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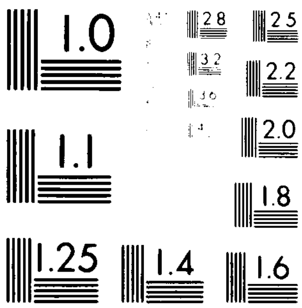

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CONCRETE AND ROCK TESTS MAJOR REHABILITATION OF DRESDEN ISLAND LOCK AND DAM, ILLINOIS WATERWAY CHICAGO DISTRICT, PHASE II COMPLIANCE SCOUR DETECTION

by

R. L. Stowe, B. A. Pavlov

Structures Laboratory
U. S. Army Engineer Waterways Experiment Station
P. O. Box 631, Vicksburg, Miss. 39180

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Drilling for laboratory testing of soil, concrete, and rock was accomplished as part of a major rehabilitation program at Dresden Island Lock and Dam, Illinois Waterway. This report covers the work accomplished during the Phase II program entitled "Compliance and Scour Detection." The purpose of the compliance work was to determine selected materials properties and to develop engineering design parameters related to materials found at the site

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that were different than previously found and tested. A secondary purpose of the compliance work was to determine the base elevation of the head-gate and tainter-gate dams. The scour detection work was accomplished to determine depth of possible scouring behind the tainter-gate dam and undercutting of the downstream apron and piers. Samples of critical clay and shale seams were obtained and tested. The base elevation of the head gates was 18 ft deeper than shown on the original working drawings, the base elevation of the tainter-gate section was 7 to 9 ft deeper than shown on original drawings. Local undercutting of the tainter-gate apron exists; however, the dam proper is not undercut. No covered scoured areas were detected behind the dam. A study of the foundation condition was made. Normal faulting was interpreted to exist beneath the lock and dam.

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PREFACE

This testing program, "Concrete and Rock Tests, Major Rehabilitation of Dresden Island Lock and Dam, Illinois Waterway, Chicago District, Phase II Compliance, Scour Detection," was conducted for the US Army Engineer District, Chicago. The work was authorized by DA Form 2544 No. NCC-IA-77-32, dated 5 April 1977, and No. NCC-IA-78-61 dated 12 June 1978.

Drilling was conducted by personnel of the Geotechnical Laboratory (GL) of the US Army Engineer Waterways Experiment Station (WES) during the periods August to September 1977, and June to July 1978, under the supervision of Mr. M. Vispi. Laboratory tests were performed at the Structures Laboratory (SL) and the GL during the periods September to November 1977 and August 1978 under the direction of Messrs. Bryant Mather, Chief, SL, and John M. Scanlon, Chief, Engineering Mechanics Division, SL. Mr. G. P. Hale supervised the laboratory testing in the GL; Mr. G. S. Wong conducted the petrographic examination. Mr. R. L. Stowe was Project Leader and was assisted in performing laboratory work at the SL by Messrs. F. S. Stewart and J. B. Eskridge and Ms. B. A. Pavlov. This report was prepared by Mr. Stowe and Ms. Pavlov.

The Commander and Director of WES during the conduct of the investigation and the preparation and publication of this report was COL J. L. Cannon, CE. Mr. F. R. Brown was Technical Director.

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CONVERSION FACTORS, U.S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U.S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
degree (angle)	0.01745329	radian
feet	0.3048	metre
feet per second	0.3048	metre per second
inch	0.0254	metre
pound (mass) per cubic foot	16.01846	kilogram per cubic metre
mile	1.609344	kilometre
pound (force) per square inch	0.006894757	megapascal
tons (force) per square foot	0.09576052	megapascal

CONCRETE AND ROCK CORE TESTS, MAJOR REHABILITATION OF
DRESDEN ISLAND LOCK AND DAM
ILLINOIS WATERWAY, CHICAGO DISTRICT
PHASE II, COMPLIANCE, SCOUR DETECTION

PART I: INTRODUCTION

Location of Study Area

1. The Dresden Island Lock and Dam is located in Grundy County, Illinois, at Mile 271.5 on the Illinois River, about 2-1/2 miles downstream of the junction of the Kankakee and the Des Plaines Rivers. The city of Joliet, Illinois, is some 14 miles upstream from the dam. Driving distance from Chicago is about 50 miles. Figure 1 indicates the location of Dresden Island Lock and Dam in relation to Morris, Illinois. The figure illustrates topographic features in close proximity to the lock and dam.

Background

2. On 11 February 1977 at the Chicago District Office, Corps of Engineers (NCC, North Central Chicago) representatives of the Waterways Experiment Station (WES) met with personnel of the NCC and the North Central Division (NCD). The WES was requested to submit a work proposal to assist the NCC in connection with a major rehabilitation study at Dresden Island Lock and Dam. The study was to be accomplished in two phases. Phase I work concerned concrete and rock exploration and laboratory testing in support of the rehabilitation (resurfacing, stabilizing lower approach wall with grouted prestressed tendons, etc.). The Phase I work is completed and results of the work are given in Reference 1. The Phase II study concerned drilling and laboratory testing to comply with certain Office, Chief of Engineers (OCE) and NCD comments as outlined in References 2 and 3. The major work effort during Phase II was to conduct

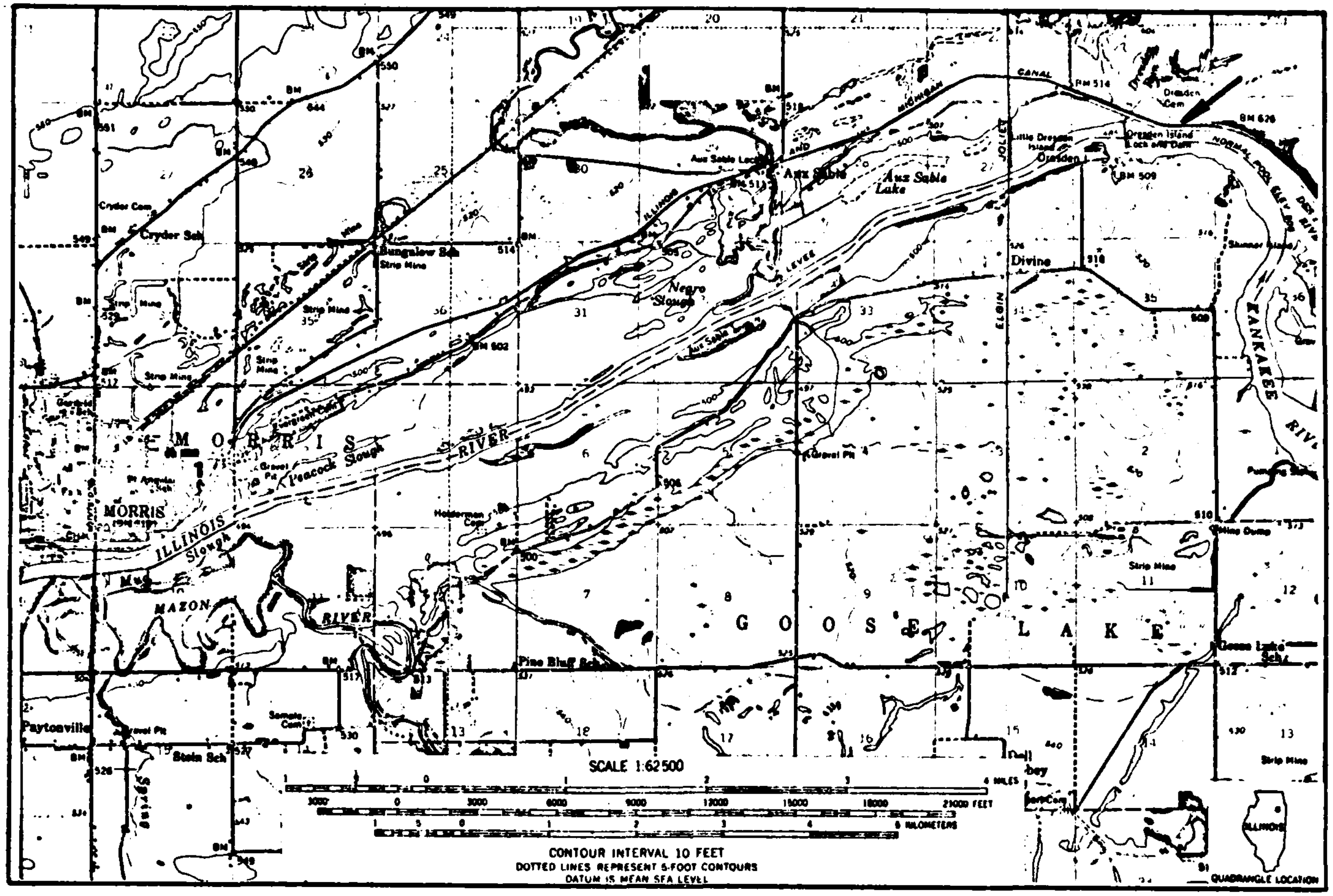


Figure 1. Vicinity map.

supplemental tests of foundation rock for purposes of developing strength envelopes based on direct shear tests. The results of these tests will be used by the District to check previous structural stability analyses if the test results are significantly lower than those previously used.

3. Subsequent to the February meeting, a scour detection study was initiated by the District as part of its ongoing investigation at Dresden Island Lock and Dam. A similar study was conducted at the Brandon Road and Starved Rock Dams. In September 1977, WES was asked to assist the NCC by drilling bedrock cores in scoured areas in back of the taintor gate dam. Careful examination, together with knowledge of the local geology, could possibly indicate boulders within or covering scoured-out holes. Previous soundings and those made in 1977 showed relatively deep scouring just downstream (D/S) of the dam. This condition exists at other dams on the Illinois Waterway having rock foundations.

4. At the completion of the Phase II and scour detection studies, recommendations were made to drill additional borings in the head gate and tainter gate sections. These borings were necessary to determine the base elevation of the dam and determine the type of bedrock beneath the dam. Borings were made and the information gathered from them is incorporated in this report. This report presents the results of the Phase II study and the results of the scour detection study described in paragraph 3. Geological information obtained from these two studies and the Phase I study was integrated with geological information gathered from references 4 through 7 to make an evaluation of the foundation condition. Attempts were made to gather additional information on the foundation condition. The State of Illinois, the NCC, and the Joliet Project Office (representatives of the operator which is the US Government), and the Illinois Geological Survey were contacted. Little geological information was obtained dating back to the time of the actual excavation for the lock and dam. Useful information was obtained from references 4, 5, and 6 and in private communications with Buschback[7] of the Illinois Geological Survey.

Objectives

5. The objectives of the Phase II study (complying with certain OCE and NCD comments) were to:

   a. Conduct limited drilling for laboratory testing of concrete and foundation rock. The rock would be tested if found to be different than rock previously observed at the lock and dam site.

   b. Make an analysis of tests conducted, a summary of the concrete condition at specific locations, and an evaluation of the foundation using available geological information.

The objective of the scour detection study was to ascertain the top of sound rock and the presence of boulders at specific locations behind the tainter gate dam.

Scope

6. The compliance and scour detection drilling (9 and 16 borings, respectively) was accomplished using a WES drilling crew, plant, and supplies. NCC Construction and Operation Division supplied the floating plant assistance for drilling on the river. A Bureau of Reclamation geologist on contract to WES logged and preserved the core for testing during some of the compliance drilling. A WES geologist performed similar duties for the remaining drilling. The core was transported to the WES as soon as feasible after drilling for each job completed.

7. The objectives of the compliance study were accomplished by drilling concrete and bedrock core and sampling earthen embankment materials and by conducting characterization property tests (unit weight, compressional wave velocities, compressive strength, and tensile strength tests) and engineering design tests (moduli, Poisson's ratio, triaxial, and direct shear tests). Direct shear tests were conducted on intact core (samples of very soft clay and shale were tested) and core containing thin clay shale/clay seams. Limited triaxial and direct shear tests were run on plastic clay (CH) obtained from the right dam abutment. Two piezometers were installed in the right dam embankment.

8. The objective of the scour detection work was accomplished by drilling and then performing a detailed examination of the bedrock for purposes of determining if the drilled rock had been recently disoriented. Information obtained during drilling was used in determining if scoured holes were present at selected drill locations.

9. A study was made to consolidate and evaluate engineering information and data, geologic and boring data, and laboratory test data as they relate to the foundation conditions at the Dresden Island Lock and Dam. Available construction and engineering data records were reviewed.

PART II: DRILLING AND EXPLORATION

Previous Exploration

10. Thirty-seven borings plus a few test pits were made by the State of Illinois in the early 1920's in connection with original construction. The borings are generally 10 ft deep and do not provide an engineering evaluation of the bedrock or the overburden; detail is lacking from the graphic representation of these borings. In general, the top of bedrock and the type of rock encountered is in good agreement with the more recent information gathered by the NCC and the WES drilling efforts.

11. In addition to the original exploration conducted by the State of Illinois, drilling was performed by the NCC in 1971 and by the WES in 1977; the WES conducted two separate drilling operations in 1977.

12. The 1971 drilling by NCC was for purposes of:

- a. Obtaining a foundation appraisal of the bedrock and backfill.
- b. Provide design parameters for use in a structural stability analysis.[4]

Four borings were put down, two in backfill adjacent to the lock structure and two just upstream of the dam structures. All borings were taken well into bedrock.

13. The WES drilling was in support of two separate projects at Dresden Island Lock and Dam. The first project was the Phase I rehabilitation (resurfacing, etc.), see paragraph 2 this report. The second project involved determining sound rock for the construction of a new mooring cell about 2000 ft U/S of the lock. During the rehabilitation study, six vertical borings in and adjacent to the lower approach wall were drilled; three borings went through the concrete wall into bedrock and three were put down in the backfill adjacent to the wall. A total of seven borings were drilled at the mooring cell sites, two through an existing cell and five next to the cell that was leaning over and almost submerged. The seven borings were taken into bedrock.

Current Drilling

14. Drilling equipment consisted of an Acker Toredo Mark II skid-mounted rotary drill rig. A D.C.D.M.A. (Diamond Core Drill Manufacture Assocation) standard 6-in. by 7-3/4-in. double tube swivel type core barrel was used with diamond bits to obtain the concrete and bedrock core. A 5-in. Huorslev-type fixed piston sampler and a 5-in. by 6-1/4-in. Denison-type core barrel were used to obtain undisturbed samples of the earth embankment material. Access to the drill holes was by a marine floating plant and for holes on top of structures by crane. Continuous samples were obtained in all borings. Eight-inch casing was set in the embankment and the bedrock when necessary to keep the borings open.

15. The compliance phase consists of nine borings. One boring (L-8) was drilled into the land lock wall through 44 ft of concrete and into 36.8 ft of bedrock. One boring (D-9) was drilled in tainter gate pier No. 6 through 49 ft of concrete and into 21.9 ft of bedrock. Two borings in the head gate section (D-45 and D-46) were drilled through 53.7 ft of concrete and about 6 ft into bedrock. Three borings in the taintor gate section (D-47, D-48, and D-49) were drilled through an average of 16.3 ft of concrete and into 25.2 ft of bedrock. E-1 and E-2 borings were drilled in the right dam abutment (an earth embankment); they penetrated 34 ft of compacted fill and alluvium and 16.6 ft of bedrock each. The types of bedrock (limestone and shale) encountered beneath the embankment and lock wall are the same types found during previous foundation explorations.

16. Generally, boring locations for the scour detection study were placed within the scoured area designated to be repaired with scour protection; see Plate A1 of Appendix A.[8] Appendix A contains the graphical representation of the scour soundings made by the NCC. A number of borings were assigned in areas where high peaks and low valleys were indicated by the scour soundings. Generally, one boring per tainter gate bay was drilled.

17. The scour detection drilling consists of 16 borings. Fifteen were drilled D/S of the tainter gates at distances ranging from 11 to

74 ft from the vertical D/S face of the tainter gate piers. Two borings were drilled through the concrete apron. The remaining boring (D-43) was drilled 100 ft D/S of the head gate structure. All scour borings with the exception of D-43 and D-36 were drilled about 10 ft deep.

18. Total footage drilled was 68.4, 156.3, and 183.1 ft of soil, concrete, and bedrock, respectively, during the compliance study, and 215.9 ft of bedrock during the scour detection study. Selected portions of the embankment material and concrete core and all of the bedrock were preserved for possible laboratory testing. Portions of the badly broken bedrock were not preserved because scheduled tests could not be conducted on the broken core.

19. Procedures for handling the field samples and preserving core for testing were the same as given in Reference 9. Color photographs of all the core recovered are included in a notebook as Exhibit A and B to this report; Exhibits A and B contain photographs of the compliance and scour detection core, respectively. Exhibit C contains the field drill core logs; the three exhibits are on file at NCC.

20. Core recovery was very good in the concrete and bedrock; core recovery, water loss, and other pertinent boring data are presented in the log of boring sheets. Except for the two borings in the embankment, all drilled holes were backfilled to their full depth with concrete prepared from a commercially available packaged dry combined mixture.

21. Two piezometers of the slotted 1-1/4-in. PVC pipe type were installed in the two borings in the embankment. One piezometer was placed U/S and one D/S of the 30-ft-long steel sheet piling running along the central axis of the embankment. The purpose of placing the piezometers on either side of the sheet piling was to determine if water was seeping beneath the sheet piling. Plates 1 and 2 present pertinent data concerning the piezometer installations. Shown in Plate 3 is a section through the embankment depicting the location of the two embankment borings DI WES E-1-77 and DI WES E-2-77. The bottom of the piezometers relative to one another are illustrated. A limited number of water level readings were taken by the drill crew after installation. The readings are presented in Plate 3.

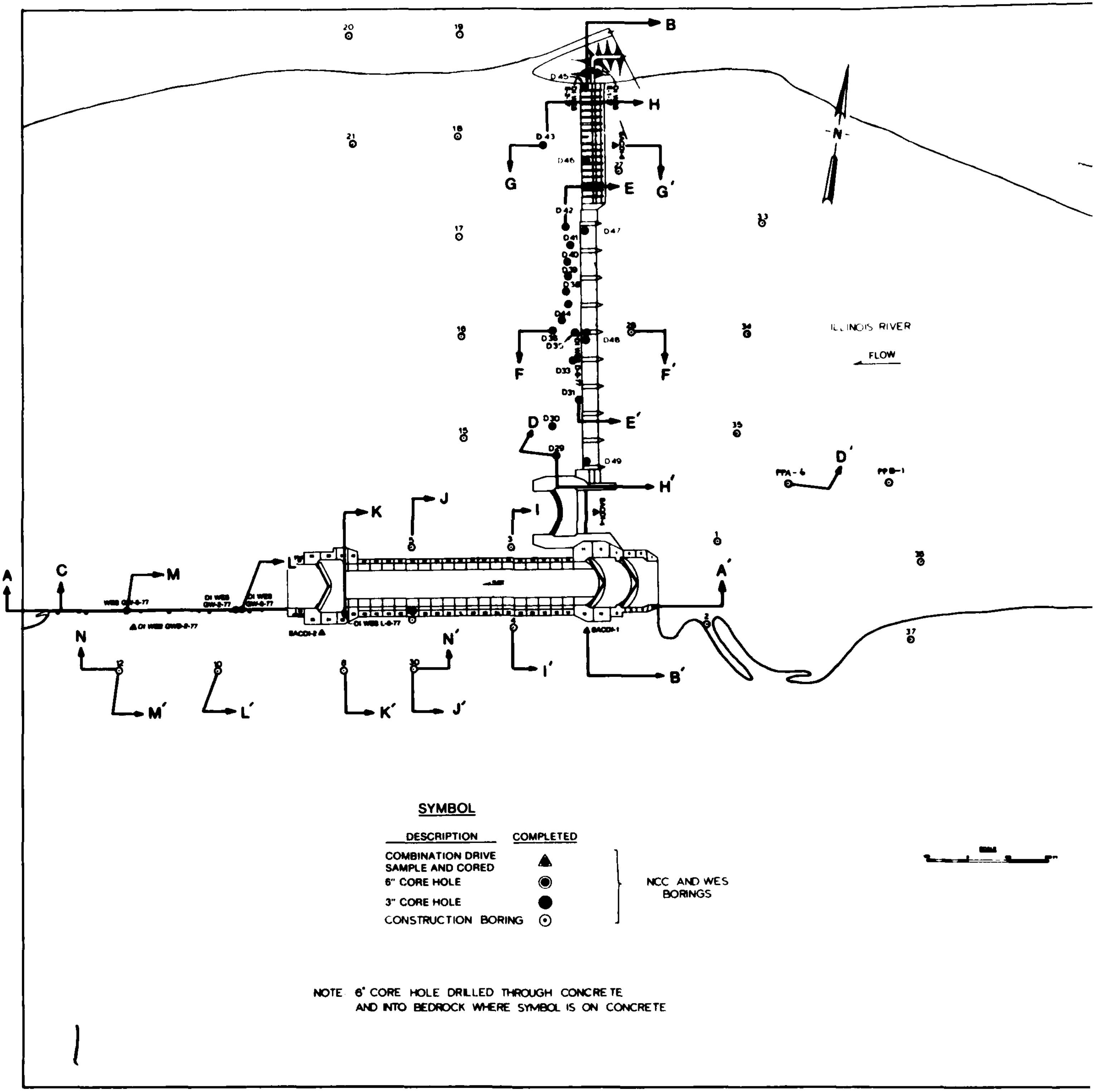
22. A number of drilling operations were performed at Dresden Island Lock and Dam. The various projects for which borings were made, the number of borings, and the boring series numbers are given in the following list:

Project (Drill Agency)	Year	No. of Borings	Boring Series No.
Connected with Original Construction (State of IL)	1921	37	1-37
Structural Stability Analysis (NCC)	1971	4	SACDI-1, 2, 4, 6
Phase I, Rehabilitation (WES)*	1977	6	DI WES GW 2, 5, 7-77 DI WES GWB 1 through 3-77
Mooring Cell (WES)	1977	7	DI WES MCA 1, 2-77 DI WES MCB 3 through 7-77
Phase II Compliance (WES)	1977	4	DI WES D 9-77, EL, 2-77 L8-77
	1979	5	DI WES D 45 through 49-79
Scour Detection (WES)	1978	16	DI WES D 29 through 44-78

*Only vertical borings considered.

23. Table 1 is a listing under each project of the above-listed borings; the State of Illinois borings are not included due to lack of information. The following information is presented for each boring: the boring type symbol, the location by structure, the elevation of the top of boring, the elevation top of rock, and the elevation bottom of boring, and the date when the boring was started. Symbols for boring types are explained in Table 1.

24. Figure 2 shows the relative location of every boring listed in Table 1. The preconstruction locations of the State of Illinois borings are presented in Plate 4 for ready reference. More precise locations for the NCC and WES borings are shown in key plan views on log of boring sheets. Log of borings is shown (in a profile) in Plates 5 through 12b, respectively, for the projects listed in paragraph 22.


ILLINOIS RIVER
FLOW
N
A A'
B B'
C
D D'
E E'
F F'
G G'
H H'
I I'
J J'
K K'
L L'
M M'
N N'
SYMBOL
DESCRIPTION
COMPLETED
COMBINATION DRIVE SAMPLE AND CORED
6" CORE HOLE
3" CORE HOLE
CONSTRUCTION BORING
NCC AND WES BORINGS
NOTE 6" CORE HOLE DRILLED THROUGH CONCRETE AND INTO BEDROCK WHERE SYMBOL IS ON CONCRETE

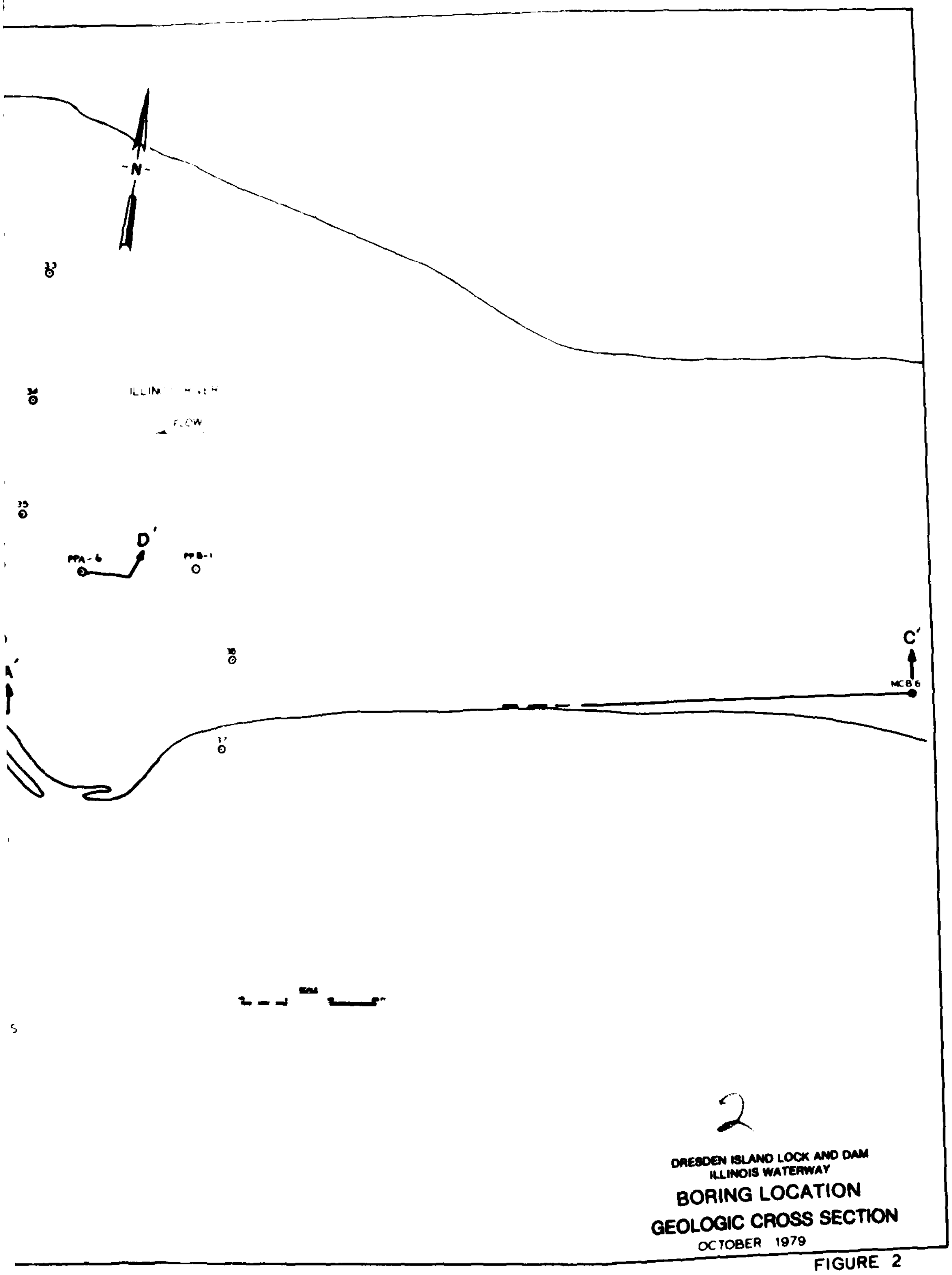

N
FLOW
D'
PPA-6
PPB-1
C'
MCB 6
2
DRESDEN ISLAND LOCK AND DAM
ILLINOIS WATERWAY
BORING LOCATION
GEOLOGIC CROSS SECTION
OCTOBER 1979

FIGURE 2

PART III: SCOUR DETECTION RESULTS

25. The scour detection borings served a two-fold purpose. First, we were able to ascertain the top of sound rock and tell if covered scoured holes existed at specific locations behind the tainter gate dam. Secondly, the geological information obtained from the borings was used along with similar information from other borings over the lock and dam in making an evaluation of the foundation. This part of the report deals with the top of rock and the scouring. Inquiries were made of the NCC and the Joliet Project Office for information concerning the possible filling of scoured areas at Dresden Island Dam. No information was found indicating that scoured holes at the dam had been filled.

26. No evidence of displaced or recently (post-dam construction) disoriented rock blocks was found during the scour drilling. The scour borings indicated porous, vuggy, and weathered limestone in 6 of the 16 holes drilled. The zone of poor rock ranged from 0.3 to 3 ft deep beginning at the top of rock.

27. As mentioned earlier, the NCC conducted a scour sounding study in 1977, and the results of the current scour detection study were compared to their results. Depth of scouring was compared whenever a WES drill site intersected a profile traverse (taken perpendicular to the dam). Overall, the scour data (top of rock) agreed with one another.

28. Three borings were intended to be drilled through the concrete apron and into bedrock. See Plate 11 for location of the three borings (D-34, D-37, and D-41). No concrete was recovered in Boring D-34 which was drilled in tainter gate bay 5. The boring was located 2 ft upstream of the apron face. Probings by hand with a metal rod were inconclusive as to the extent of the apron adjacent to D-34. Top of rock was encountered about 8 ft (El 470.9 ft) below the design elevation of the top of the apron. Boring D-37 in gate bay 6 revealed 0.4 ft of concrete apron with a 6-ft void beneath. The extent of the void area under the apron is not known. The logs of borings D-34 and D-37 show that the top portion of these holes contain weathered vuggy rock. The missing portion of the apron in bay 5 and the undercutting of the apron in bay 6

suggest heavy scouring behind Gates 5 and 6. The significance of the scoured area in terms of stability of the dam will be discussed in Part VII. Boring D-41 was drilled through 1.2 ft of concrete and into sound bedrock. A tight bond between concrete and bedrock was found in D-41.

PART IV: GEOLOGICAL CHARACTERISTICS

Backfill and Embankment

29. The backfill behind the lower approach wall and the land lock wall is described in References 1 and 4. Briefly, the backfill is probably spoil from the lock and dam excavations and consists of a mixture of silt, clay, sand, gravel, and boulders. The material did not appear to be laid down in an orderly fashion.

30. A connector dike (earthen embankment) links the head gate section with the toll path of the Illinois-Michigan Canal. The material in the dike, as revealed in the 34 ft uncovered in each of Borings E-1 and E-2, consists of compacted fill made up of mixtures of clay, sand, gravel, and a small amount of silt. Within the sampled area, content of these materials may reach as high as 90 percent clay, 85 percent gravel, or 70 percent sand. The largest percentage of clay occurs in the top 20 ft, the largest percentage of gravel in the middle 7 ft, and the largest percentage of sand in the last 7 ft.

Bedrock Stratigraphy

31. A columnar section of the Cincinnatian Series of Ordovician aged rock found in the general vicinity of the lock and dam is presented in Figure 3. The bedrock revealed by borings at the lock and dam belongs to the Ordovician aged Maquoketa Group. The topmost unit encountered is the Ft. Atkinson Limestone Formation. The Ft. Atkinson Limestone is a light gray coarse grained, argillaceous limestone. For the most part it is a competent, dense limestone, but it is vuggy and in places becomes very porous. The limestone is fossiliferous, crinoid stems being most abundant, and it is pyritic. It contains many lenses and nodules of green clay and green shale up to 1 in. thick, the average beginning about 1/4 in. The limestone tends to break along these clay and shale coated bedding planes every 0.1 to 1.0 ft. The bedding planes containing the clay and shale are termed filled partings in this report. Asperities

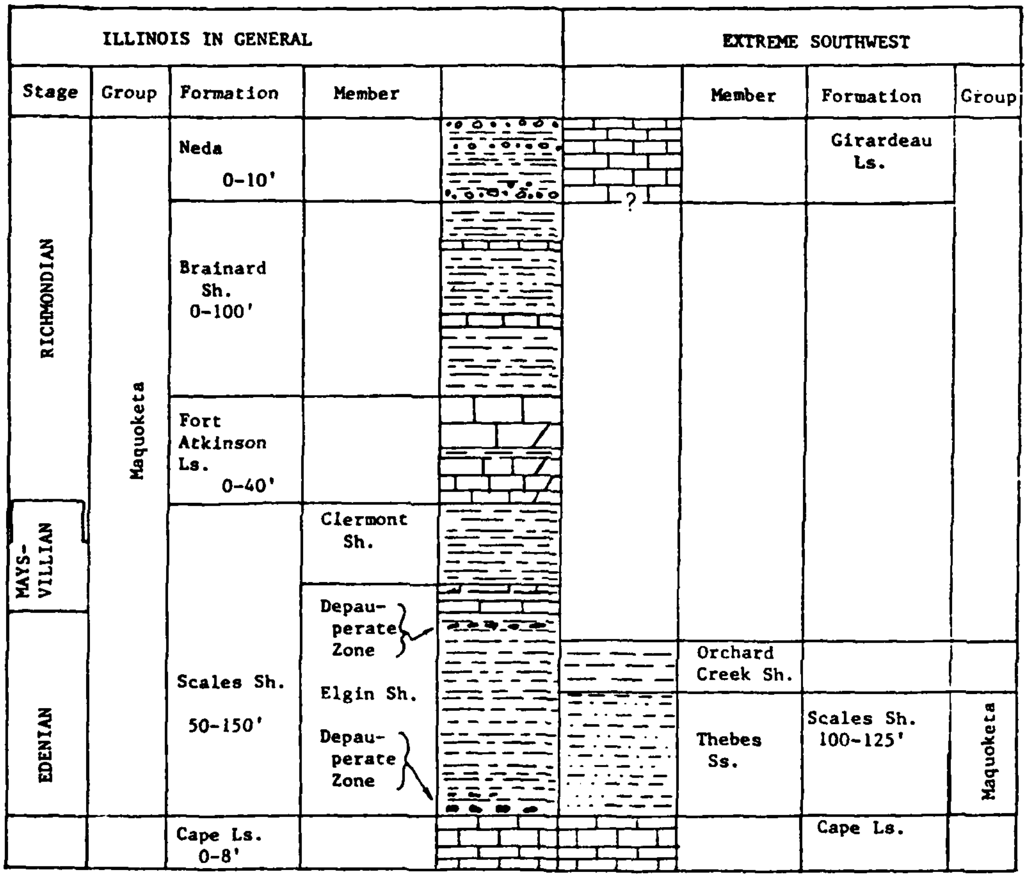


Figure 3. Columnar section of the Cincinnatian Series of the Ordovician System (after Templeton and Willman, 1963), see reference 5

along the bedding are generally interlocking, about 1/4 to 3/8 in. high with 3-in. periods, and rather flat peaks and valleys. Coating of the bedding surfaces is from 50 to 100 percent. Below the Ft. Atkinson is the Scales Shale Formation. The Scales is a dark brown to gray dense, uniform shale which slakes readily upon exposure into 0.1- to 0.5-ft disks. Up to 10 percent of the rock is dolomitic with several areas reaching a composition of a shaley dolomite. Fossil content is low with the exception of a "fossiliferous zone" found in the deeper borings. This zone was referred to in Reference 1 as a depauperate zone; recent studies indicate a fossil zone. The word depauperate refers to "falling short of natural development;" thus, the fossils in this zone are smaller than would be expected.

32. The contact of the limestone and the brown shale is a green shale unit. This unit was denoted as a shaley clay in the Rehabilitation report;[1] a more detailed description was made possible by examination of 14 additional samples from the scour detection and compliance borings. The shale is heterogeneous, composed primarily of a compact green shale within which are found thin limestone layers, thin green clay layers, and a number of fossils found in layers, as well as randomly sprinkled through the shale. These are angular limestone fragments found at the top of the unit in borings D-35, D-38, and D-42. The most noticeable and important element in the shale unit is a green plastic clay layer located near the top of the shale. This clay layer is the same color and consistency as the clay found along bedding planes in the limestone and in the green shale. The thickness of the green shale ranges from 0.6 to 1.8 ft with average thickness of 1.3 ft. Thickness of the clay unit ranges from 0.05 to 0.34 ft, averaging 0.15 ft. The green clay layer, in several instances, contains angular limestone and shale pieces. This may indicate some movement at the clay layer which caused fragmentation of the shale and limestone.

Bedrock Structural Characteristics

33. The nearest major regional structure is the Sandwich Fault Zone. The zone is located about 6 miles northeast of Dresden Island Lock and Dam. It trends NW-SE and is about 85 miles in length. Maximum displacement at the center of the fault is 800 ft.[11]

Faulting adjacent to the lock and dam

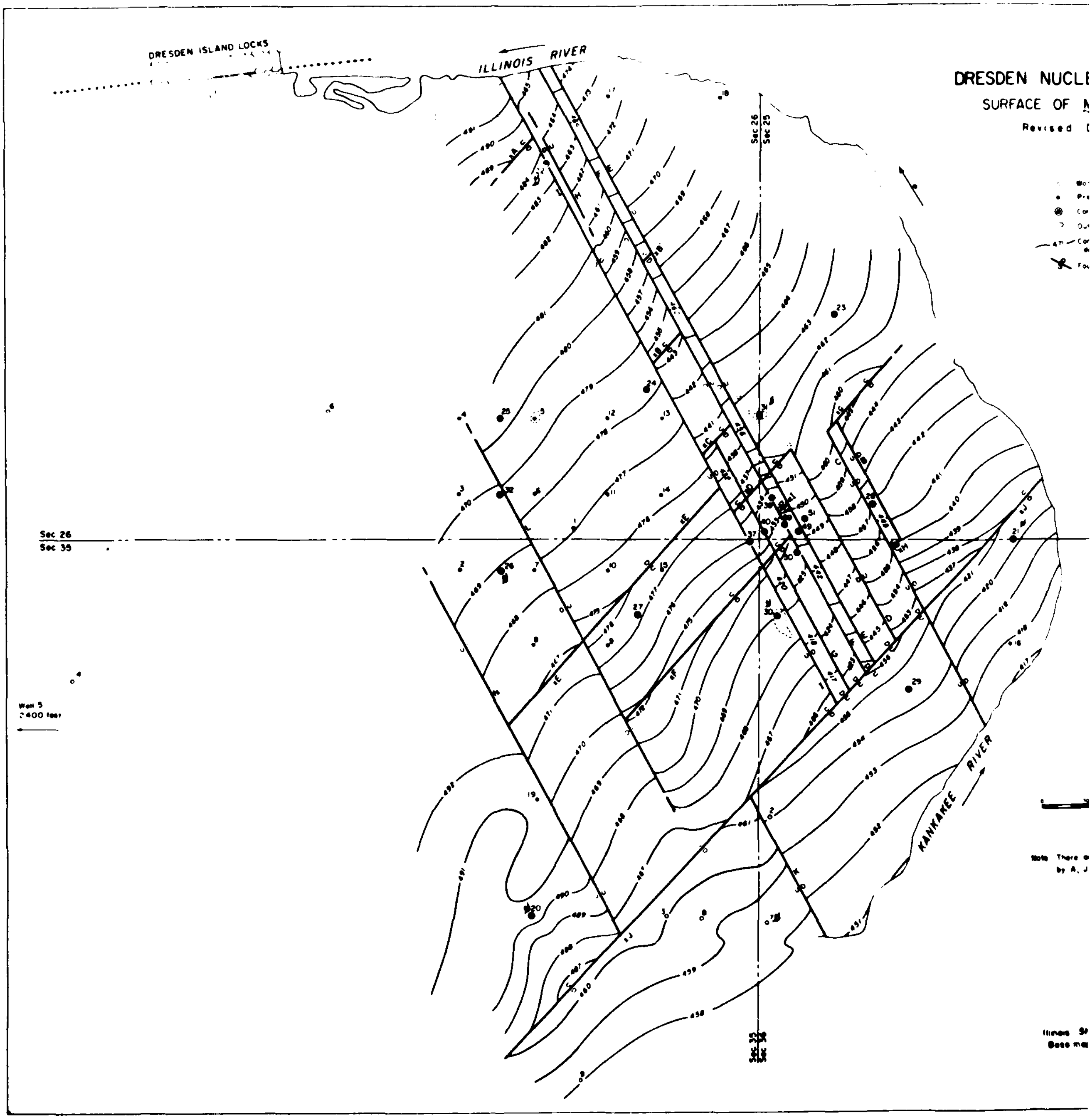
34. In 1956 the Illinois State Geological Survey (ISGS) conducted an analysis of the geologic conditions at the site of the Dresden Island Nuclear Power Plant; records and cores of wells, drill holes, and core borings were used in the analysis.[12] Structural contour maps and cross sections that relate graphically their analysis of the geologic conditions at the site show numerous intersecting faults with maximum displacements of 50 ft. The faults trend N 28° W and N 43° W; the N 28° W faults roughly parallel the Sandwich Fault Zone. The nuclear power plant is adjacent to and within 0.5 miles of the Dresden Island Lock and Dam.

35. Direct evidence of faulting was observed in only three borings studied by the ISGS. However, in attempting to determine the probable extent and magnitude of the evident faults, the Survey found discrepancies in the top elevation of the Maquoketa shale that could only be satisfactorily explained as the result of faulting. Figures 4 through 7 are copies of the Survey's maps and cross sections that show the orientation, extent, frequency, and throw of faults.

Faulting at the lock and dam

36. During the study to consolidate and evaluate engineering information, an aerial photograph of the Dresden Island Lock and Dam proved interesting. Figure 8 shows the lock, a part of the dam, and a portion of the nuclear power plant. The northwest trending line across the open pasture between the power plant and lock is believed to be the surface expression of a fault. A similar orientated fault line was interpreted by the Illinois Geological Survey; see Figure 4, fault L.

37. Figure 9 contains photographs taken a few hundred feet from the lock looking toward the lock. The field geologist reported that the feature seen in these photographs is a low relief ridge with stepped-like


DRESDEN ISLAND LOCKS
ILLINOIS RIVER
Sec 26
Sec 25
Sec 26
Sec 35
Sec 35
Sec 36
Well 5
2400 feet
KANKAKEE RIVER

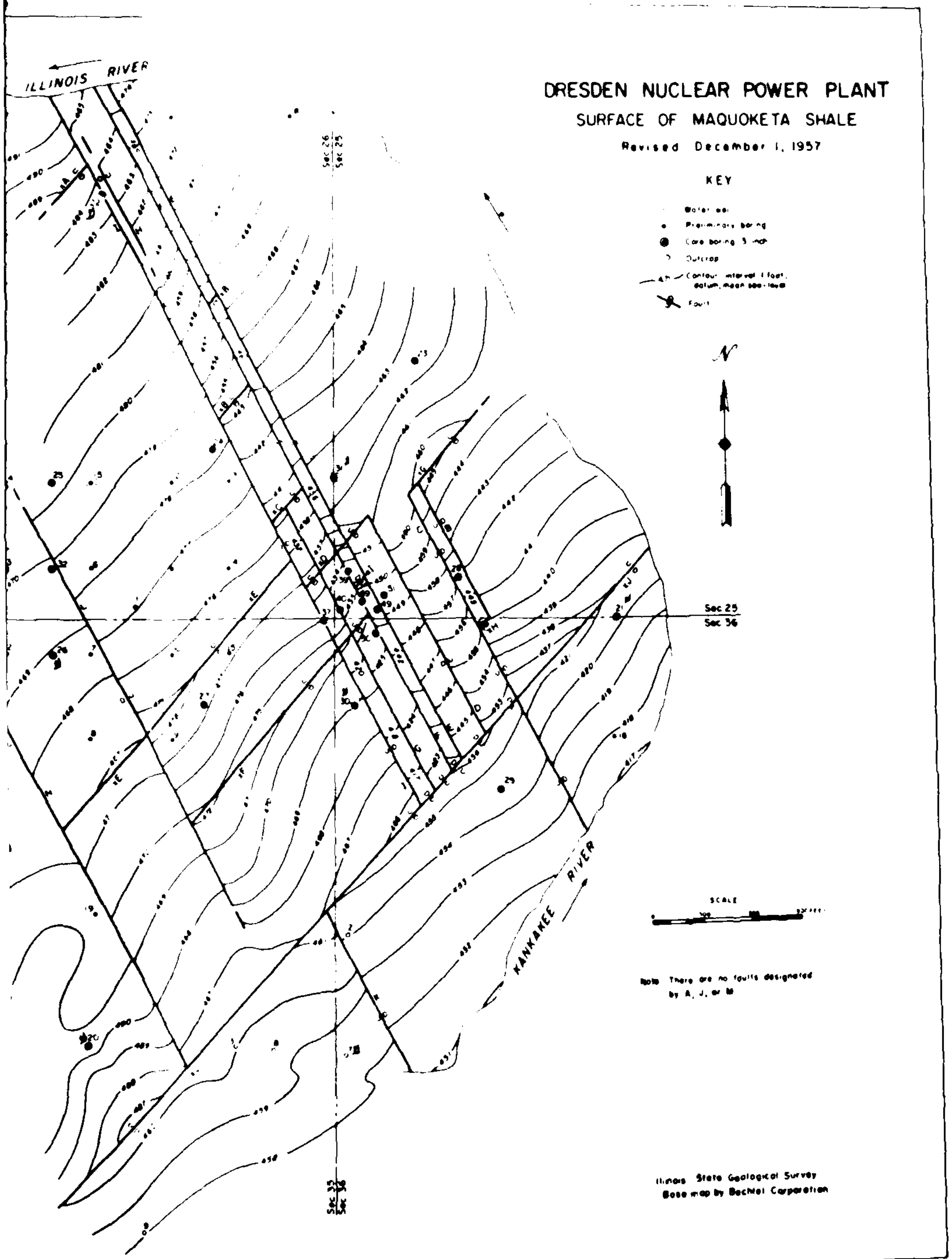
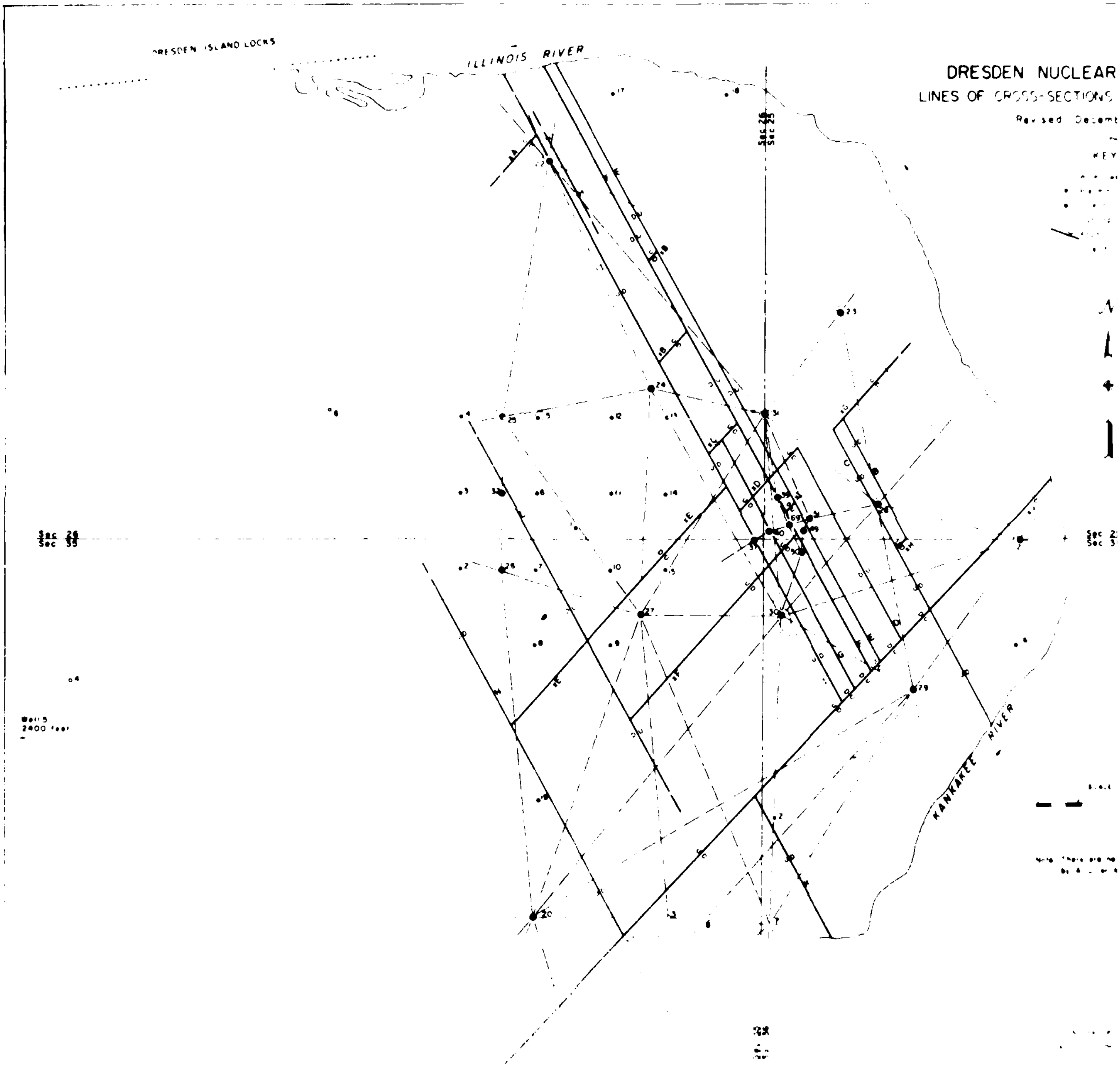

DRESDEN NUCLEAR POWER PLANT
SURFACE OF MAQUOKETA SHALE
Revised December 1, 1957
KEY
Preliminary boring
Core boring 3 inch
Outcrop
Contour interval 1 foot, datum, mean sea-level
Fault
N
ILLINOIS RIVER
KANKAKEE RIVER
Sec 26
Sec 25
Sec 25
Sec 36
Sec 35
Sec 36
SCALE
Note: There are no faults designated by A, J, or M
Illinois State Geological Survey
Base map by Bechtel Corporation

FIGURE 4


DRESDEN NUCLEAR
LINES OF CROSS-SECTIONS
DRESDEN ISLAND LOCKS
ILLINOIS RIVER
KANKAKEE RIVER
Sec 26
Sec 25
Sec 26
Sec 35
Well 5
2400 feet

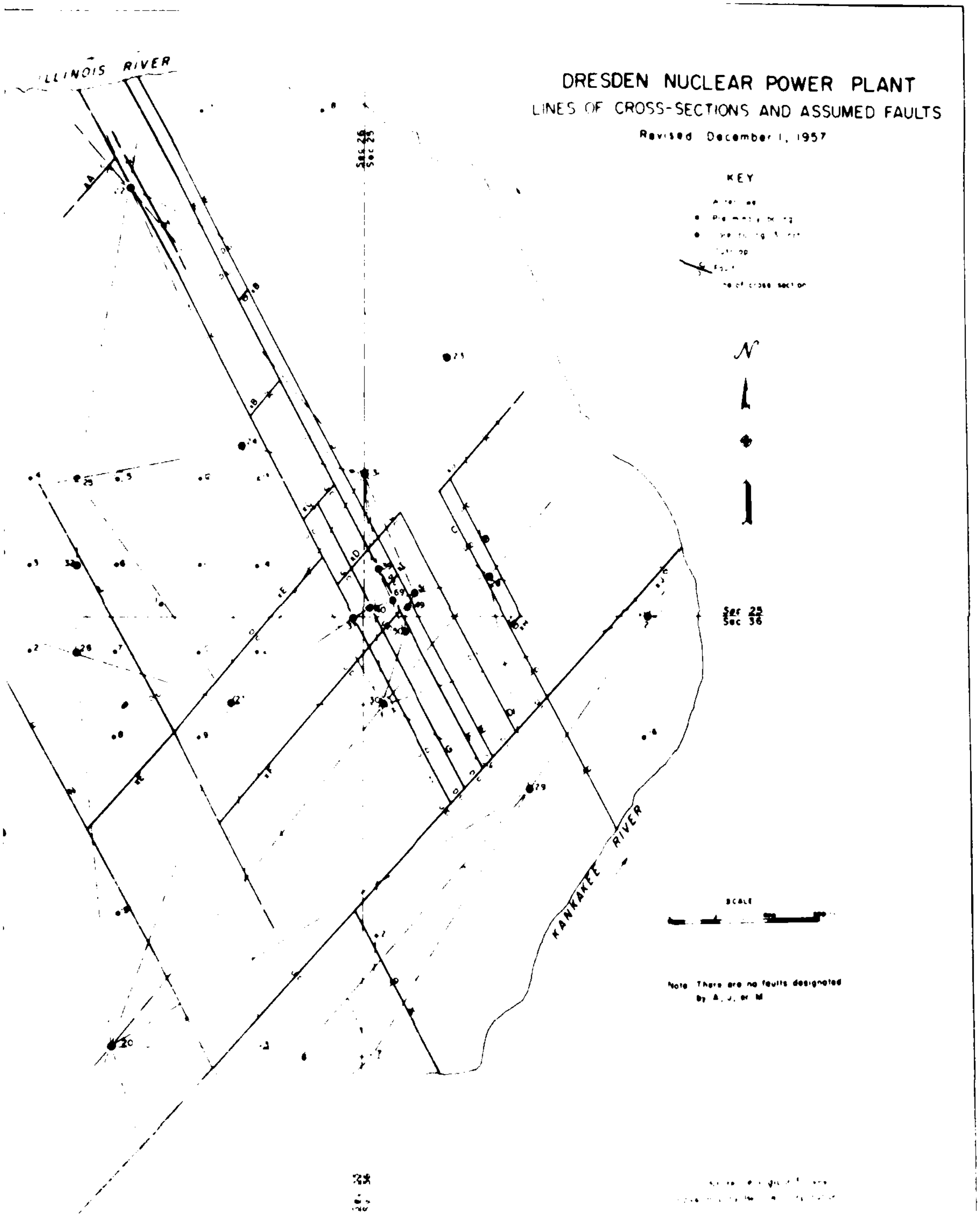

ILLINOIS RIVER
DRESDEN NUCLEAR POWER PLANT
LINES OF CROSS-SECTIONS AND ASSUMED FAULTS
Revised December 1, 1957
KEY
Sec 26
Sec 25
N
Sec 25
Sec 36
KANKAKEE RIVER
SCALE
Note: There are no faults designated by A, J, or M

FIGURE 5

2

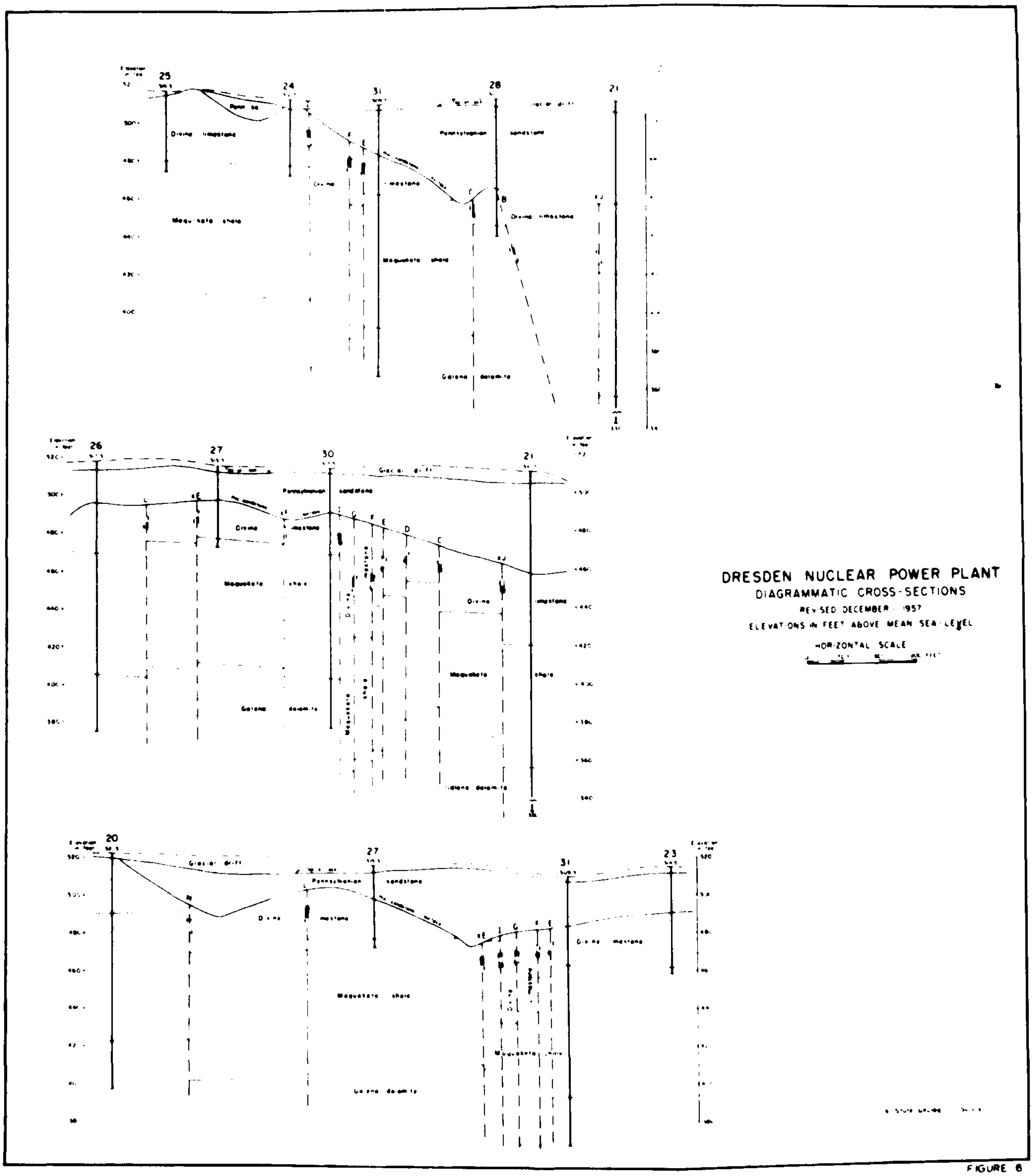

25
24
31
28
21
Glacial drift
Pennsylvanian sandstone
Divine limestone
Maquoketa shale
Galena dolomite
26
27
30
21
Glacial drift
Pennsylvanian sandstone
Divine limestone
Maquoketa shale
Galena dolomite
20
27
31
23
Glacial drift
Pennsylvanian sandstone
Divine limestone
Maquoketa shale
Galena dolomite
DRESDEN NUCLEAR POWER PLANT
DIAGRAMMATIC CROSS-SECTIONS
REVISED DECEMBER 1957
ELEVATIONS IN FEET ABOVE MEAN SEA LEVEL
HORIZONTAL SCALE

FIGURE 6

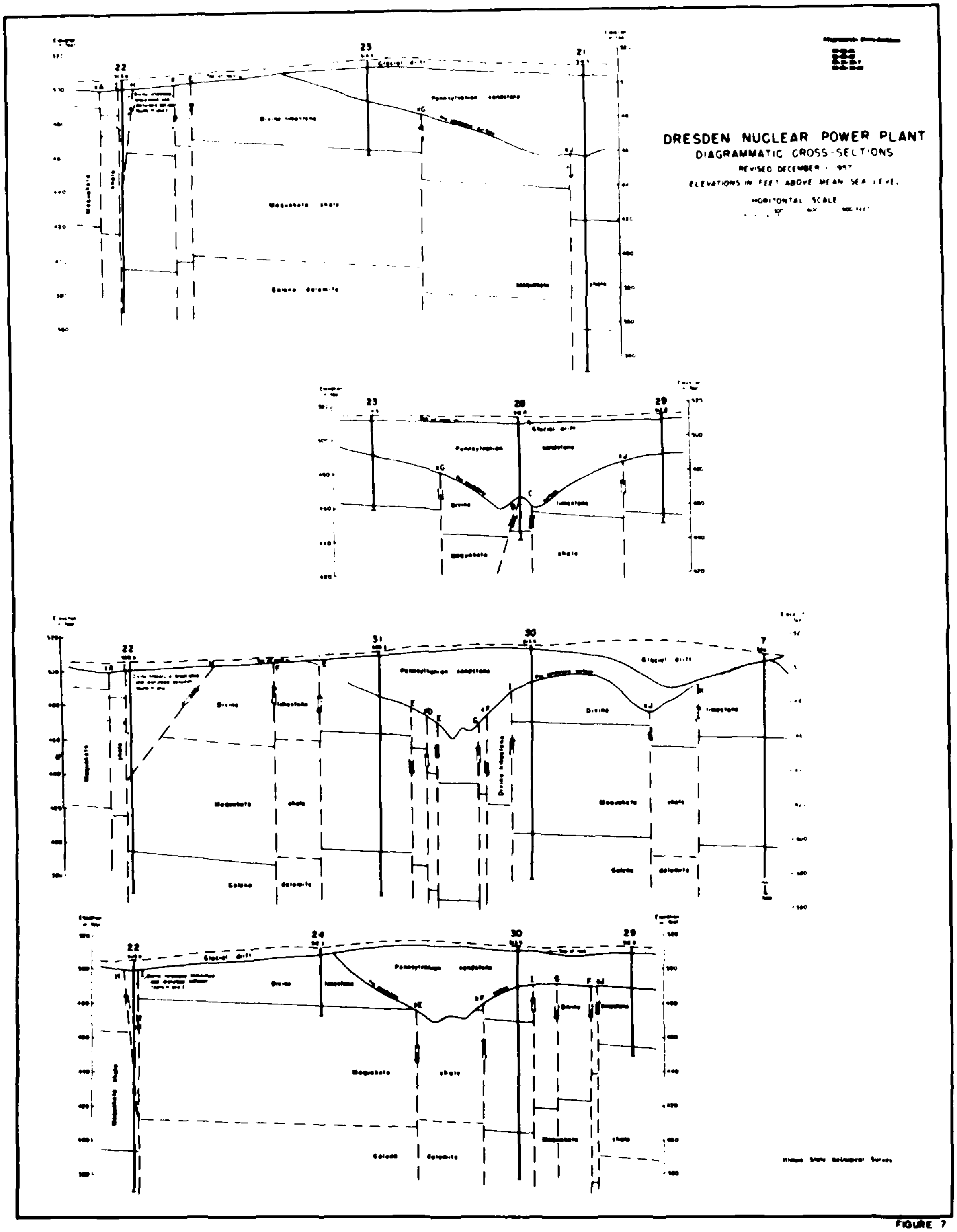
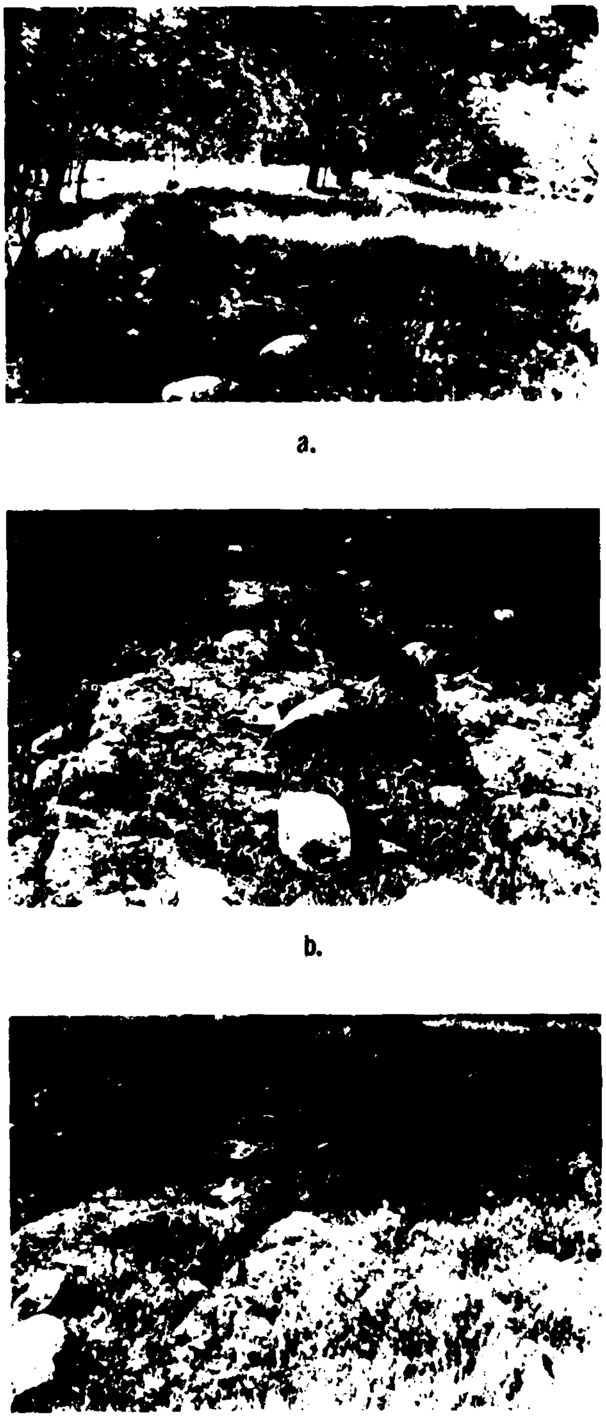

DRESDEN NUCLEAR POWER PLANT
DIAGRAMMATIC CROSS-SECTIONS
REVISED DECEMBER
ELEVATIONS IN FEET ABOVE MEAN SEA LEVEL
HORIZONTAL SCALE
22
23
21
28
29
31
30
7
24
Glacial drift
Pennsylvanian sandstone
Divine limestone
Maquoketa shale
Galena dolomite
Illinois State Geological Survey

FIGURE 7

Figure 8. Air photograph, Dresden Island Lock and Dam and Dresden Nuclear Power Plant.



a.

b.

c.

Figure 9. (a) Looking southeast along ridge, (b) and (c) Stepped appearance along ridge.

slopes at about 30°. Weathered Ft. Atkinson limestone outcrops on the surface with numerous intersecting joints paralleling the northeast and northwest trending faults described at the nuclear power plant site. Trenching in the area would be required in order to ascertain if the ridge resulted from faulting. However, trenching is not recommended due to the strong evidence, interpretation of boring information by the ISGS, and the air photograph that the structural feature between the power plant and the lock is a fault.

38. The geology at the lock and dam is much more complex than assumed during the 1971 study; the 1971 study had only four borings from which to make a general assessment of the foundation.

39. The green shale contact of the Ft. Atkinson limestone and the Maquoketa shale was referenced by Buschback[7] as a good marker bed. The green shale varies over the lock and dam site from El 480.2 to El 446.0 ft. The variation is not gradual nor does it proceed from one edge of the site to the other.

40. A map of the surface of the Maquoketa shale and 14 cross sections were drawn in an attempt to explain the discrepancies observed in the marker bed elevations; see Plates 12c through 19e. It was assumed that the regional dip of the strata, as determined from records of wells in the area,[12] was fairly uniform southeasterly. Knowing that faults were observed adjacent to the lock and dam, reversals or other departures from the assumed regional dip which were evident by the elevation of the marker bed, were interpreted as discrepancies due to faults. The assumed faults were then inserted at locations that would most simply account for the discrepancies. The faults were assumed to be vertical because the boring information and site topography revealed no evidence to the contrary. The authors concur with the following quote from Reference 12, given as a warning in using the map and cross sections. The statement originally pertained to the faulting at the nuclear power plant but applies equally to the faulting at the lock and dam.

> "Consequently the map and the cross sections must be used with the realization that (1) the existence, position, trend, inclination, and throw of the faults are largely interpretative... (2) some of the faults may not exist at all, (3)

> others may exist but occur at somewhat different positions or have different trend, inclination, and throw, and (4) there may be additional faults as well as fractures without displacement that are not revealed by our interprelative analysis. It was further assumed that the faulting occurred at some time subsequent to the deposition of the Divine limestone and prior to the deposition of the Pennsylvanian sandstone, as no evidence of faulting in the sandstone has been noted, but this assumption may also be proven erroneous."

41. Sixteen faults have been interpreted at the lock and dam site. The fault orientations were chosen to parallel the N 28° W and N 43° W faults reported at the power plant. The contour spacing is similar to the spacing developed for the nuclear power plant tract. However, in several cases the contour interval changes more rapidly, indicating a stepper tilting block. The greatest discrepancy in the marker bed is 18 ft.

42. Reference 13 presents strong evidence that the faulting at the lock and dam is related to the Sandwich fault zone. Faults at both locations have the same northwest to southeast trend and probably occurred at the same time. The structural significance of the inferred faults at the lock and dam cannot be fully assessed. The Sandwich fault and those studied on the power plant tract were reported to have been inactive for millions of years. Movement along the faults in the Dresden Island area appears unlikely since they have been inactive since Pennsylvanian time.

43. The presence of slickensides, brecciated rock, or vertical jointing in five borings is additional supportive evidence of faulting; see Plate 12c for location of the five borings. Two of the five borings, containing such evidences of faulting, are adjacent to interpreted faults. Twenty-one borings had nominal 0.3-ft zones of badly fractured rock without a preferred orientation; spacing of the fractures was < 4 in. The zones occurred in the limestone and in the underlying shale; however, the limestone contained the majority of fractured zones. The majority of the joints were inclined at greater than 70°. Joints below the concrete structures were encountered about 10 ft below the concrete.

44. Possible weak zones in the foundation rock include the plastic clay and shale-filled bedding planes in the limestone, the green plastic clay layer (average thickness is 0.15 ft) in the green shale unit, clay-filled partings in the green shale unit, and the normal faults. The clay- and shale-filled bedding planes in the limestone are not traceable as individual units in the foundation rock. They do exist throughout the thickness of the limestone and occur across the entire site. Locally they could participate in failures. The green plastic clay layer at the top of the green shale unit, while not a continuous layer over the site, occurred at different elevations in most borings, the exception being those borings through the head and tainter gate dam sections and the land lock wall. The broken and seamed limestone and the clay layer apparently were removed at these locations during construction. The clay layer was recovered from about 5 ft beneath the base of the lower approach wall. Locally the clay layer could participate in failure of the approach wall. The clay-filled partings in the green shale (they exist just below the green plastic clay layer) would probably not control horizontal failure within the green shale unit; the green plastic clay layer would. Green plastic clay, similar to the clay found in the green shale unit, is found associated with slickensides, badly fractured rock, and brecciated rock.

Geologic Cross Sections

45. The locations of the 14 cross sections selected to illustrate the foundation conditions at the lock and dam are shown in Figure 2. The cross sections were drawn incorporating selected borings from the six projects listed in paragraph 22. The following tabulation gives the cross sections, general location of section, borings in sections, and plate number of sections.

Section	General Location	Borings in Section*	Plate No.
A-A'	Lock and lower approach wall	GW-2, GW-5, GWB-2, GW-8, L-8, SACDI-1 and 2, 4, 29	13
B-B'	Head and tainter gate dam	E-1, E-2, SACDI-1, 4 and 6, D-9, D-45 through 49	14
C-C'	Approach wall to U/S mooring cell	A-A' borings plus MCB-6, MCA-1	15
D-D'	Arch dam	D-29, SACDI-4, PPA-3	16
E-E'	D/S of tainter gates	D-31 through 33, D-35, D-38, D-39, D-40 through 42, D-44	17
F-F'	Tainter gate dam	D-9, D-34 through 36	18
G-G'	Head gate dam	SACDI-6, D-43, D-46	19a
H-H'	D/S head and tainter	D-29, D-30, D-36, D-43	19b
I-I'	Lock wall	4	19c
J-J'	Lock wall	5, 29, 30	19c
K-K'	Lock wall	L-8, SACDI-2, 5	19c
L-L'	Lower approach wall	GW-2, GWB-3, 10	19d
M-M'	Lower approach wall	GW-5, GWB-2, 12	19d
N-N'	Lock wall backfill	8, 10, 12, 30	19e

GW, guide wall equals approach wall; B, backfill, SACDI, Stability Analysis Civil Dresden Island; E, embankment; MC, mooring cell; L, lock; D, dam; PP, protection pier.

*The first five letters and the last two numerals have been omitted from the WES boring numbers for simplicity.

The detailed location of the borings can be seen in the plan view at the top of each plate. Note that the profiles are nonlinear since they incorporate borings placed near to as well as on the lock and dam structures. These sections give a good overview of the bedrock beneath the entire installation.

46. Throughout the cross sections, the green shale unit, containing the average 0.15-ft thick clay layer, can be seen at different elevations under the lock and dam structures. The unit is present under a portion of the lower approach wall; this was the only area where the clay

was detected beneath a concrete structure. Plate 16 indicates that the green shale unit could exist beneath the right gate wall of the future lock if the wall was built as shown on the working drawings. With the removal of this shale unit in the head and tainter gate sections and lock wall, it is reasonable to believe that the unit was also removed before placement of the right gate wall of the future lock.* A future lock was originally intended to be built sometime after construction of the existing lock and dam. A thin arch dam was constructed between the taintor gate dam section and the river wall of the existing lock; the future lock has not been built and the arch dam remains as constructed in the 1920's.

47. The cross sections show the interpreted faults. The fault traces are projected from the structural contour maps and drawn on the cross sections where they intersect the cross section lines. Note that the fault traces are not always shown intersecting the marker bed at the boring-fault intersection. Top of rock was taken from boring information, and the reader will have to visualize top of rock between borings not on the cross section line through concrete.

48. The short dashed lines, as shown in Plate 14, are probably as-built extensions to construction drawings based on actual boring data. Note that beneath the dam the limestone and the green shale has been removed. It was not detected in any of the six borings through the dam structures. Working drawings specify that the base of the dam be taken to the limestone shale contact. The base of the tainter gate section has been lowered 7 to 9 ft while the head gate section has been lowered about 18 ft. A typical founding elevation for the tainter gate section

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* After this report was written and prior to publication, an NX boring (DI WES D-50-79) was drilled through the right gate wall (28 ft U/S of the D/S face of the wall) of the future lock. The dark brown to gray dense shale unit was encountered beneath the concrete. Top of hole elevation is 510 ft, bottom of concrete elevation is 464.9 ft, and bottom of boring is 462.0 ft. The drilling log for this boring is on file at the Corps Office, Rock Island District. The Rock Island District is currently assigned to oversee the Illinois Waterway, and hence the reason for sending the boring log to Rock Island.

is suggested to coincide with the highest elevation where the base was detected by borings (El 465.0). The base is founded on the competent dark gray to brown shale.

Bedrock Weathering

49. The upper 3 ft of the Ft. Atkinson limestone has been subjected to solution by the ground water in the area. The approximate 3-ft zone is quite vuggy and weathered. The bedrock just downstream of the dam section appears to have been affected most; the poor rock can be seen in 6 of 16 borings downstream of the dam. The weathered bedrock was not found beneath the head and tainter gate dam sections nor in the one boring through the land lock wall. The weathered rock was apparently removed during construction. Weathered bedrock was found beneath a portion of the lower approach wall.

PART V: TEST SPECIMENS AND TEST PROCEDURES

Cores Received

50. Concrete and rock core from 4 and 16 borings, compliance and scour detection studies, respectively, were received at the WES. Pertinent information concerning the core received for the compliance study is presented in Table 2, and similar information is presented in Table 3 for the core received from the scour drilling operation. Upon receipt of the core at WES, the boxes were stored in the laboratory for three days, after which time test specimens were selected and the specimens placed in the concrete moist curing room until time for testing.

Selection of Test Specimens

51. A detailed visual examination of all the core was made to assist in the selection of representative test speciemens of concrete and of the different foundation materials. Concrete test specimens from the two deep vertical borings (D-9 and L-8, tainter gate pier and land lock wall, respectively) were selected from the top, middle, and bottom of the core. No concrete was tested from the core recovered during the scour detection drilling program. Test specimen depths shown in the tables of test results represent the mid-section of the test specimen; i.e., El 509.1 is the mid-point of a piece of core from El 509.6 to 508.6. Six-inch diameter by twelve-inch long concrete and rock cores were used for testing, the exception being the specimens for direct shear testing. Characterization properties; effective (wet) unit weight (γm), compressional wave velocity (Vp), and compressive strength (UC), and engineering design properties; Young's Modulus (E), and Poisson's ratio (ν) were determined.

52. For the engineering design tests (modulus of elasticity, Poisson's ratio, triaxial and direct shear) an attempt was made to select test specimens to be representative of the rock in close proximity to the base of the structure. The test assignment locations can be obtained from appropriate tables of test results.

53. There were three types of specimens tested in direct shear: concrete cast on rock, intact, and filled partings. Cores of shale and clay were tested as intact specimens and the filled parting specimens contained clay as the filler material. For purposes of this report clay-filled partings are synonymous with clay seams. Filled partings in limestone and shale were tested.

54. During the major rehabilitation testing, samples of the critical foundation materials (clay seams in limestone and shale/clay) could not be obtained for testing.[1] It is believed that an adequate number of these two materials have been obtained from the borings put down during the compliance and scour detection studies. Shear tests have been conducted on these two materials and it is believed that the test results adequately characterize the shearing resistance of the materials.

Test Procedures

55. The characterization properties tests and the engineering design properties tests were conducted in accordance with the appropriate test methods tabulated below:

Property	Test Method
Characterization	
Effective Unit Weight (As Received), γ_m	RTM 109*
Dry Unit Weight, γ_d	RTM 109
Water Content, w	RTM 106
Compressional Wave Velocity, Vp	RTM 110 (ASTM D 2845)**
Compressive Strength, UC	RTM 111 (ASTM D 2938)
Tensile Splitting Strength, T_s	ASTM C 496-71
Engineering Design	
Elastic Modulus, E	RTM 201 (ASTM D 2148)
Direct Shear Strength	RTM 203
Multistage Triaxial Strength	RTM 204
R Triaxial Strength	EM 1110-2-1906[10]

* Proposed Rock Test Method, Corps of Engineers, in review prior to publication.

56. The concrete-on-rock specimens were prepared for direct shear testing in accordance with procedures as described in Reference 9. No petrographic examinations of concrete or rock were considered necessary during the conduct of the compliance or scour detection studies.

PART VI: TEST RESULTS AND ANALYSIS

Concrete

57. The concrete core recovered from Borings L-8 and D-9 is in very much the same condition as the core previously recovered during the major rehabilitation (Phase I) drilling program. See pages 22-26 in Reference 1 for a general description. Boring L-8 was drilled in the land lock wall and the core revealed that the top 0.4 ft of concrete was new. The new concrete was placed during a previous resurfacing of the top of the lock wall. From 0.4 ft to 1.3 ft the concrete was highly deteriorated and was recovered as gravel in the core barrel. The remaining 43 ft of concrete is structurally sound and should serve its original intended purpose. The interface between the concrete and the shale bedrock was well bonded. The concrete core recovered from Boring D-9 in tainter gate pier No. 6 was deteriorated in the top 0.4 ft with the remaining 48.6 ft in excellent condition. The bond between the concrete and shale bedrock was tight. The concrete within the central portion of pier No. 6 is structurally sound. See Reference 1 for a description of the surface concrete in the tainter gate piers. The concrete from D-9 and L-8 contained no significant void areas and was well consolidated. Six concrete cores were tested and the test results are described in the following paragraphs.

Characterization Properties

58. The results of the characterization property tests and the engineering design tests are presented in Table 4 for the concrete and bedrock. The average value, the range, and the number of tests for the concrete, limestone, green clay, and shale are tabulated below.

Summary of Characterization Properties

	Effective Unit Weight γ_m, lb/ft^3	Dry Unit Weight γ_d, lb/ft^3	Water Content w, %	Comp Wave Velocity Vp, ft/sec	Compressive Strength UC, psi
Concrete					
Average	151.3	141.1	7.2	15,553	6,190
Range	3.8	4.6	2.6	1,080	1,490
No. of tests	6	6	6	6	6
Limestone					
Average	170.8	167.3	2.1	17,992	6,280
Range	8.2	10.0	1.2	4,070	10,610
No. of tests	4	4	4	4	7
Shale (gry to brn)					
Average	157.3	149.3	5.4	9,123	2,440
Range	5.0	6.3	1.5	1,830	1,050
No. of tests	3	3	3	3	3
Shale (grn)					
Average	163.7	155.8	5.1	--	--
Range	18.8	19.8	1.8		
No. of tests	6	6	6		

59. An analysis of the characterization properties will be presented for each of the above materials.

a. Concrete. The unit weights are reasonable and consistent with the unit weights reported in Reference 1 for cores taken from the land lock wall. Previous average values of 150.0 lb/ft^3 for concrete at a depth of 3 ft compare well with the average value of 151.3 lb/ft^3 presented in the above tabulation. The average compressional wave velocity for the previously tested core from horizontal borings in the land wall is 15,258 ft/sec. This value compares well with the similar value of 15,553 ft/sec. The average compressive strength presented above, 6190 psi, compares well with a similar value of 5860 psi reported in Reference 1. The range in the unit weights, velocities, and strengths indicates uniformity of the internal concrete at the lock and dam. See Reference 1 for a description of the near surface concrete.

b. Limestone. The unit weights of the limestone are reasonable and consistent with similar values reported in Reference 1. The average is 170.8 lb/ft^3 and compares well with the average of 176.5 lb/ft^3 reported previously. The average velocity is 17,992 ft/sec and is reasonable and consistent with the similar value of 18,004 ft/sec reported previously.[1] The strength of the limestone varies greatly from a low of 2,380 psi to a high of 12,990 psi. The large range is due to the variable condition of the limestone. Portions of the rock recovered are quite porous, contain clay nodules, and are weathered. These portions generally exhibited the lower strengths. The competent limestone exhibited the higher strength. The poor limestone was not present in Borings L-8 and D-9. The weathered limestone was probably removed during construction of the lock and dam. However, it is present under sections of the lower approach wall. The weathered limestone would not be expected to fail in compression considering the present static structural loadings. Failure would likely occur as a shear failure in the limestone along clay seams. This assumption will be discussed in greater detail later under Sliding Friction.

c. Shale. The average unit weight of the gray to brown shale, 157.3 lb/ft^3, compares well with a similar value, reported in Reference 1; the previous value is 155.4 lb/ft^3. These values indicate a moderately dense, compact shale. The velocity obtained on the shale perpendicular to bedding averaged 9123 ft/sec which compares well with the previous average value of 8778 ft/sec. The velocities recorded for the three specimens indicate consistency within the sample. The data might be useful in correlations with in-situ seismic velocities if such were available. The average compressive strength is 2440 psi compared to 1230 psi for similar shale previously reported.[1] The previously tested shale was recovered from under the lower approach wall (Boring GW-2) at about El 460.0 ft. The shale reported herein was recovered from under tainter gate pier No. 6 at about El 457.0 ft. The distance between the two locations is about 1000 ft. Differences in the strength of shale from the same formation is not uncommon. The difference could be caused by slight compositional makeups of the rock or slightly different cementitious agents. This shale is from a formation that contains areas of calcareous shale as distinct layers and some areas that are not readily distinguishable. The water content of the shale tested during this study is about 1.5 percent lower than the shale previously studied. The difference in water content could easily account for the differences in strength. The lowest strength, 950 psi (68.4 tsf), was measured on core from GW-2 and this value is suggested for use in design.

Modulus of Elasticity and Poisson's Ratio

60. Results of the modulus of elasticity and Poisson's ratio tests are presented in Table 4. The stress-strain relation for the concrete, limestone, and gray to brown shale is presented in Plates 20-23. The modulus was calculated as an incremental value between 500 and 1000 psi on the stress-strain curve; in most cases this stress increment corresponds to the linear portion of the stress-strain curve for both rocks. The curves for both rocks exhibit an initial concave upward portion which is associated with closure of stress relief cracking caused when the rock was removed from the foundation.

61. The average modulus for the concrete, limestone, and shale is 4.25×10^6 psi, 3.20×10^6 psi, and 0.69×10^6 psi, respectively. The values for the concrete and limestone are in good agreement with the previously reported moduli.[1] The modulus for the shale is about twice the value reported in Reference 1. It is not unreasonable to expect the modulus to be higher considering that the strength of the shale reported herein is about twice the strength as given previously.

62. The average Poisson's ratio for the concrete and shale is 0.25 and 0.20, respectively. Both these average values are reasonable considering compressive strength values.

Triaxial

63. The results of the multistage triaxial tests are presented in Plate 24 as Mohr's stress circles. The procedure for specimen preparation, testing, and for determining the cohesion and the joint friction angle (sliding friction) is given in Reference 9. The sliding friction value from the multistage triaxial test could be used for computing structural stability for that portion of the lock walls or dam sections directly above bedrock. Although the contact between concrete and bedrock was tight (excellent bond existed) in the boring through the land lock wall and No. 6 tainter gate pier, the sliding friction value allows for a structural stability analysis to be made using a lower bound value

of friction. The sliding friction value (ϕ_j) for concrete-on-shale (gray to brown) is 20 degrees with cohesion being zero. The value appears reasonable for a low to moderately hard shale; a low to moderately hard shale is considered to be between 1000 to 2500 psi compressive strength.

Maximum and Residual Shear Strength

Maximum and residual shear stress criteria

64. The following discussion of shear stress criteria is taken from Reference 14 and is followed in this report.

65. Designers are commonly interested in the maximum available shear strength. The maximum shear stress points are identified as τ_{max} in Figure 10. The maximum shear stress usually corresponds to the peak of the shear stress versus displacement plot (curve a of Figure 10); however, some confusion may arise where strain-hardening is encountered. When strain-hardening occurs, an initial peak usually occurs at a relatively small displacement, followed by an increase in shear stress to a value greater than the initial peak. When this happens, the first peak is termed the maximum shear stress corresponding to initial failure and the latter is the ultimate shear stress.

66. If the residual shear strength is to be determined, then displacement is continued until the shear stress approaches the horizontal asymtotic value of residual shear stress τ_R (curve a of Figure 10). When the zone tested exhibits only a residual shear strength, curve b of Figure 10 may be obtained. In such cases, the maximum shear stress attained is the residual shear strength. By testing a number of specimens, each at a different normal load, the maximum and residual strength failure envelopes are developed by plotting maximum and residual shear stresses versus corresponding normal stresses.

Multistage shear tests

67. The multistage shear method was used to test the filled parting specimens during the early part of this study. It was necessary to

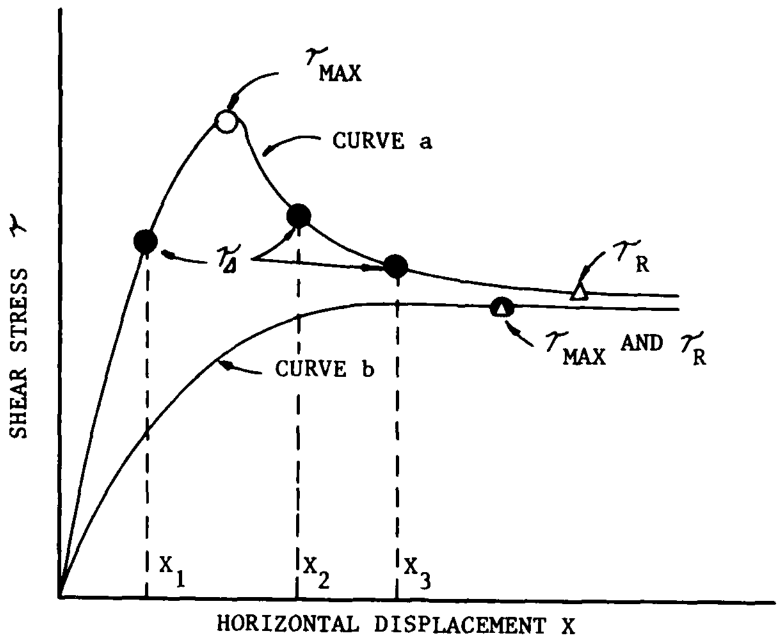

Figure 10. Maximum and residual shear stress, and displacement failure criteria, after Zeigler.

do this because only a limited number of test specimens were available during the Phase I and Phase II work. Additional filled parting specimens were obtained from the site (mainly the river bank) and were tested using the standard direct shear method. All other specimens were tested using the standard direct shear method.

68. In the repetitive tests the specimen is initially sheared until failure is obtained. The shear and normal load is released, and the shear block repositioned and resheared under an increased normal load. Repetitive testing can be useful; however, the test results are difficult to interpret, since after initial shearing the discontinuity surface undergoes irreversible changes that affect the shear strength values from succeeding tests. The failure envelopes obtained from repetitive testing will usually define strengths intermediate between the maximum and residual strength. In general, repetitive testing will yield a conservative estimate of the maximum shear strength. It is recognized that repetitive testing may be used to measure the residual shear strength. Continued displacement ultimately reduces the strength

available along the discontinuity to its residual shear strength. After a number of shearing cycles, the maximum stress is, in fact, the residual stress.

69. The shear strength values obtained from the multistage testing of filled partings were used in the Chicago District's stability analysis; hence, conservative shear strengths were used. These early values are tabulated below for easy reference. The lock and dam are stable against sliding when the conservative strength values are used.

Phase I and II Multistage Shear Strengths

Filled Parting	Limestone	Green Shale	Green Clay
Green Clay	c = 2.7 tsf ϕ = 8.9°	c = 2.0 tsf ϕ = 18.5°	c = 2.0 tsf* ϕ = 9.3°
Shaley Clay	c = 4.2 tsf ϕ = 35.5°		

* Intact specimens of the green clay layer.

Direct shear test results and discussion

70. Two types of direct shear tests were conducted to determine maximum strength of intact specimens and sliding friction characteristics of discontinuous specimens. Maximum strengths were measured for intact shales, shale and limestone containing concrete on rock interfaces (concrete bonded to rock), and clay; residual strengths were obtained where available. Sliding friction properties were measured for specimens along either precut surfaces, clay, and shale/clay filled partings. The direct shear test results of intact specimens are present in Plates 25-34; shear stress values, load-deformation curves, and normal versus shear deformation curves are presented. The direct shear test results from the discontinuous specimens tested as filled partings are presented in Plates 35-47, and tested as precut specimens in Plates 48-50. Some plates contain just the shear stress values or load-displacement curves or both. The direct shear test results from the

S-test on the green clay layer are presented in Plate 51. Profiles of specimens with clay and shale/clay seams in limestone, seen after testing, are presented in Plates 52 and 53. Maximum and residual strength failure envelopes for the intact, precut, and concrete on rock specimens are presented in Plates 54-60.

71. The failure envelopes for the critical zones within the foundation are presented in Figures 11 through 13. The critical zones in terms of sliding are the green clay layer at the base of the Ft. Atkinson and the clay and shale/clay filled partings in the Ft. Atkinson. The bedrock feature having the lowest residual strength is the green clay layer; residual strengths of $c = 0$ and $\phi_r = 12.5^o$ were obtained; see Figure 11. A conservative envelope was constructed by placing it through the lower data points and then shifting it to have a zero cohesion. The green clay layer exists beneath a section of the lower approach wall. It was not found beneath any other concrete at the site.

72. As mentioned earlier, multistage shear tests were conducted on filled parting specimens out of necessity. Direct shear tests were run for comparison. The direct shear testing shows that maximum strengths are higher than reported for the multistage tests. Maximum and residual strengths were obtained on the filled partings using the following test technique. Four specimens each were used for the two differently filled partings. Each of the four specimens had different normal loads applied; 1.5, 2.5, 4.0, and 8.0 tsf. A specimen was consolidated with a normal load, sheared, and a peak load determined. The shear and normal loads were removed, the specimen repositioned, the same normal load reapplied and sheared again. This sequence was continued with the same specimen until a residual strength was obtained. Another specimen was likewise tested at a greater normal load, and so forth. The peak and residual shear stress values from the four specimens were used to construct maximum and residual strength failure envelopes; thus, maximum and residual phi angles were determined; see Figures 11 and 12.

73. The shear stress-shear deformation curves were generally characteristic of the two curves presented in Figure 10. It will be noted that on the majority of the shear stress-shear deformation plots,

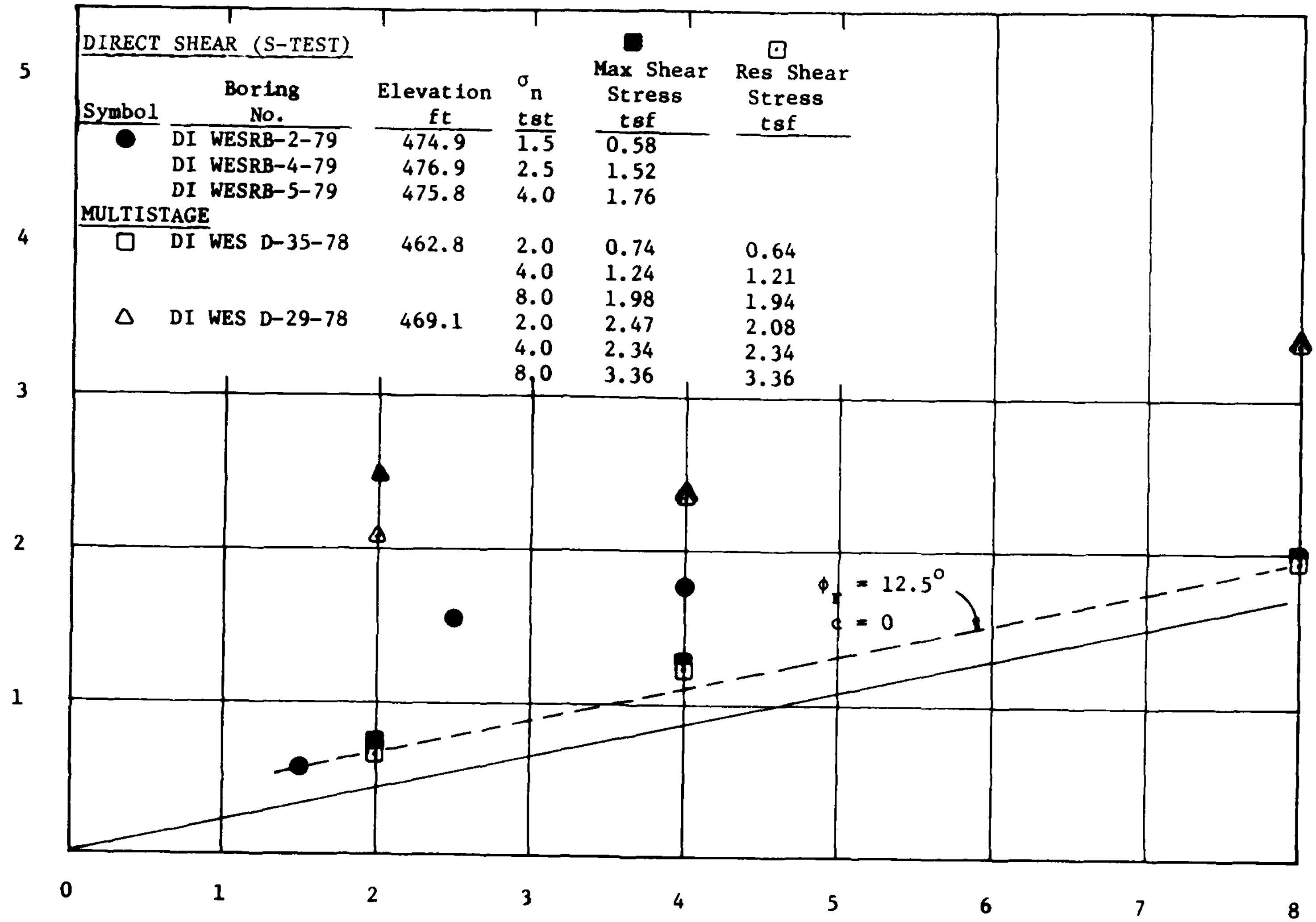

Figure 11. Direct shear (S-Test) and multistage shear test results, green clay layer at contact of Ft. Atkinson & Maguoketa Formations

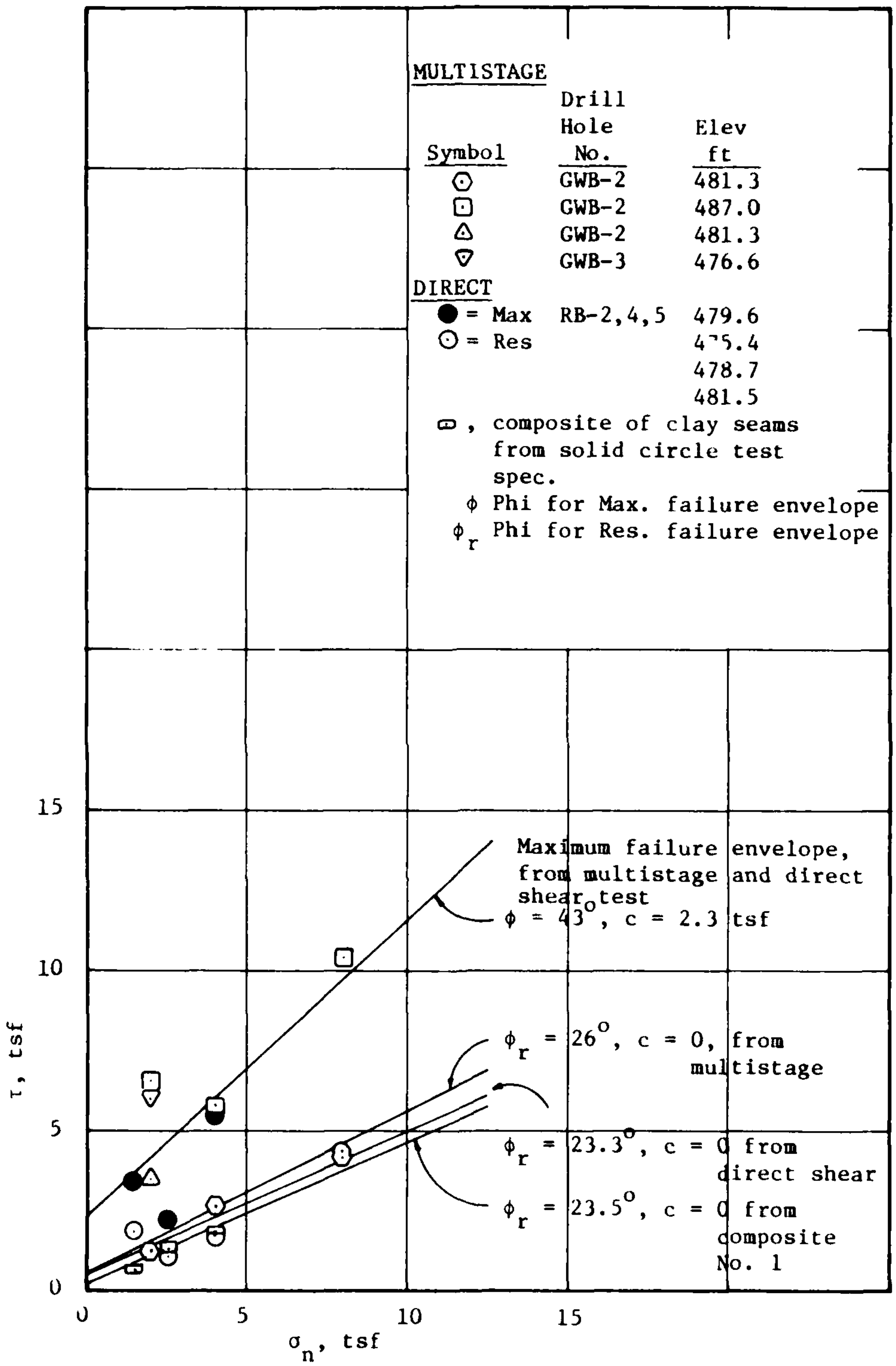

Figure 12. Direct shear results, clay seam in limestone.

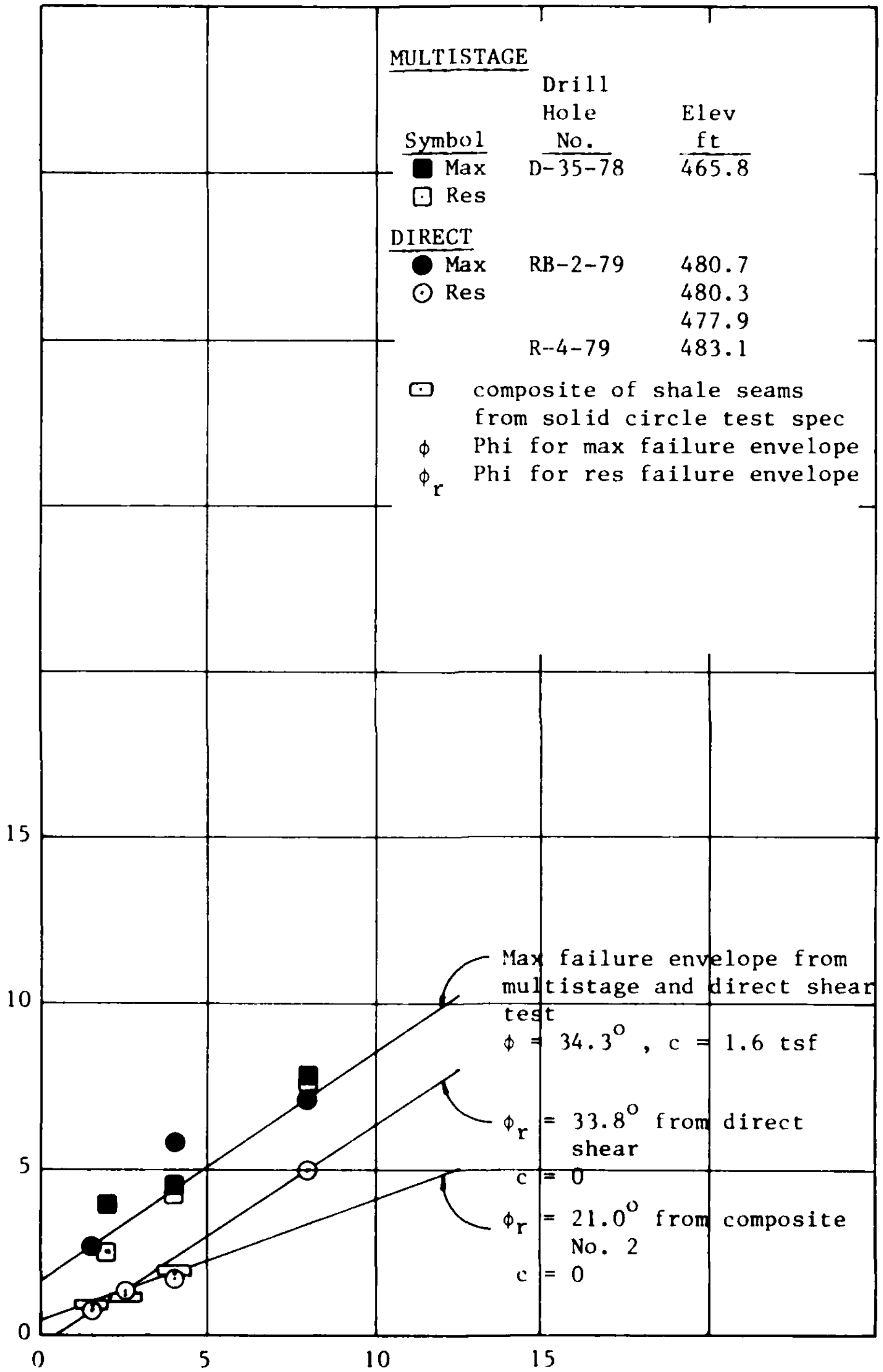

Figure 13. Direct and multistage shear results, shale/clay seam (1/8 to 1/4 in. thick) in limestone.

the curves look as if the specimens underwent strain hardening. Between 0.1- and 0.2-in. deformation, the curves turn sharply upwards. Some strain hardening probably has occurred for some of the specimens. However, it is believed that the rather sharp increase in shear stress is due to a combination of things. Some of the tests were set up with too small a gap between the shear blocks, causing the blocks to bind together during a test. And with some of the tests, the bonding agent holding the specimens in the shear blocks was set too high and came into contact during testing. The test data are not in question for those curves that do not rise sharply at about 0.2 in. of deformation and bewteen zero and about 0.2 in. of deformation.

74. Additional tests were run to verify that the residual shear stress values selected from the load-deformation curves did in fact represent a residual state of stress. The residual shear stress was picked where the load-deformation curve was horizontal or approached the horizontal asymtotic value of residual shear stress; this stress value generally coinsided with the sharp upwards turn of the curve.

75. The clay and then the shale/clay material was scraped from the filled parting specimens after direct shear tests were run. Composite specimens were made and tested in a soils 1- by 3- by 3-in. direct shear device. The shear stress values from the composite specimens were used to plot failure envelopes; see Figures 12 and 13. For the clay seam specimens, the shear strength parameters from the composite specimens agree well with the residual shear strength from the direct shear tests. Figure 13 shows that the composite specimens yielded shear strength values less than obtained from the direct shear tests of the shale/clay partings.

76. It is recommended that when the clay and shale/clay seams are considered for design the following shear strengths be used: clay, maximum and residual, respectively, be $\phi = 43^o$, $c = 2.3$ tsf, and $\phi_r = 23.3^o$, $c = 0$; shale/clay, maximum and residual, respectively, be $\phi = 34.3^o$, $c = 1.6$ tsf, and $\phi_r = 21^o$, $c = 0$.

Embankment

77. The results of triaxial and direct shear tests are presented in Table 5. The peak stress values, the stress circles, stress-axial strain plots and certain characterization properties are presented in Plates 61 and 62 for an undisturbed soil sample from Boring E-1. A few undisturbed samples of plastic clay (CH) were obtained from the embankment. One $\overline{R}$ triaxial test was conducted on a plastic clay (CH) recovered from El 494.8 ft. Two direct shear tests were conducted on specimens of the same plastic clay material. The specimens came from within a foot of El 494.8 ft. The direct shear results are presented in Plates 63 and 64. The $\overline{R}$ and direct shear test results are not considered to be representative of the soil in the embankment because too few tests were conducted. Other materials like the lean clays, clayey sands and gravels were not tested. Testing of a large number of soil samples in order to arrive at representative design parameters was beyond the scope of work. The data are considered preliminary. The average unit weights, angle of friction, and cohesion are summarized below for the plastic clay (CH).

$$\gamma_{wet} = 111.8 \text{ lb/ft}^3$$
$$\gamma_{dry} = 82.6 \text{ lb/ft}^3$$
$$\gamma_{sub} = 52.1 \text{ lb/ft}^3$$
$$c' = 0.0$$
$$\phi' = 31^\circ$$

PART VII: SUMMARY OF FOUNDATION CONDITION, CONCRETE, RECOMMENDED DESIGN VALUES

Scour Detection

78. Inquiries were made into the possibility that scoured holes behind the tainter gate dam were filled with riprap or armor stone. To our knowledge this has not been done at Dresden Island Lock and Dam.

79. A total of 16 borings were drilled downstream of the dam. No evidence of displaced or recently (post-dam construction) disoriented rock blocks was detected during the scour drilling. The core recovered from these borings revealed about 5 ft of seamed and broken limestone; the top 3 ft was weathered and quite porous. The remaining rock recovered consists of green and gray to brown shale. The green shale contained the 0.15-ft average thick clay layer that was found to exist over the site. In places the elevation of clay layer coincided with or lay close to the base elevation of the dam as shown on contract drawings. Overall, the scour data (top of rock) agreed well with the scour profiles compiled by the NCC.

80. Three borings were located in order to drill through the concrete apron. No concrete was recovered in Boring D-34 (gate Bay 5). Top of rock in this boring was encountered 8 ft below the design elevation of the top of the apron. The other two borings reached concrete near design elevation. Only 0.4 ft of concrete was recovered in D-37 (gate Bay 6) with a 6-ft void beneath. The third boring revealed a good concrete apron/bedrock contact. The missing portion of apron in Bay 5 and the undercutting in Bay 6 suggest severe scouring behind Bays 5 and 6.

Foundation Condition

Backfill and embankment

81. The backfill behind the lower approach wall is probably spoil from the lock and dam excavation. It consists of a mixture of silt, clay,

sand, gravel, and boulders. The greater majority of the boulders are the same rocks as found in the core recovered at the site. The embankment material in the right dam abutment is compacted fill consisting of a mixture of clay, sand, gravel, and a small amount of silt.

Bedrock stratigraphy

82. The bedrock encountered through drilling at the lock and dam belongs to the Ordovician aged Maquoketa Group. The topmost unit is the Ft. Atkinson Limestone Formation. The limestone is coarse-grained, argillaceous, generally dense except for small highly porous zones. It is fossiliferous and pyritic. The limestone contains seams and nodules of green clay and shale up to 1-in. thick; the lateral extent of the lenses is believed small. Bedding planes are generally interlocking. The Scales Shale Formation underlies the Ft. Atkinson. The shale is dark brown to gray and dense.

83. The contact of the limestone and brown shale is a green shale unit with an average thickness over the site of 1.3 ft. The shale is heterogeneous, composed mainly of shale within which are thin limestone, green clay, and fossil layers. Angular limestone fragments are occasionally found at the top of the unit. A green plastic clay layer averaging 0.15-ft thick is located near the top of the unit.

Geologic cross section

84. The cross section indicates the contact between the Ft. Atkinson limestone and the Scales shale. The cross sections give an overview of the bedrock material and show the variation in thickness of the Ft. Atkinson limestone and the green shale contact unit between the limestone and the Scales shale. The location of weak clay partings within the limestone and shale and the clay layer at the top of the green shale unit can be readily detected which should be beneficial in conducting of a structural stability analysis.

Bedrock structural characteristics

85. The Sandwich Fault Zone is the closest major regional structure to the lock and dam. The zone is located about 6 miles to the northeast of the lock and dam. The fault zone is about 85 miles long and

trends NW-SE; maximum cumulative displacement is about 800 ft at its midpoint in southeastern DeKalb County.[11]

86. The ISGS study conducted to the south and within 0.5 miles of the lock and dam revealed numerous intersecting normal faults. The faults are oriented N 28.5°W and N 43°E. The study site is the location of the Dresden Island Nuclear Power Plant. The majority of the faults were interpreted; however, direct evidence of faulting was observed in borings and trenches. Ongoing investigations by the ISGS indicate the faulting is present to the north of the Dresden Island Lock and Dam site. Air photographs and a study of the ground surface between the power plant and the lock indicate the presence of a fault between the two installations.

87. Boring information, including original construction borings, shows that similar foundation conditions are present at the lock and dam as were found at the power plant. The green shale unit at the top of the Maquoketa Scales shale was used as a marker bed for correlating strata at the lock and dam site. The unit varies in elevation from 480.2 ft to El 446.0 ft. The variation is not gradual nor does it proceed from one edge of the site to another. The discrepancies in elevation are attributed to faulting. Sixteen faults have been interpreted at the lock and dam site; 11 are believed to exist beneath the concrete structures; see Plates 12c through 19e. The fault orientations parallel to the northwest and northeast faults described at the power plant. The maximum displacement of the interpreted faulting is 17 ft and occurs in an area near the lock. The faulting at the power plant and the lock and dam are reported to have been inactive for millions of years. Movement along these faults appears unlikely since they have been inactive since Pennsylvanian time.

88. The majority of the joints observed at the lock and dam were nearly vertical. Vertical fractures were present in the limestone, and some fractures contained limestone breccia. A few thin horizontal clay seams within the green shale unit contained angular pieces of limestone while others contained limestone and shale. Slickensides were observed

on relatively low angle (25^{o}) fractures in the gray to brown shale. The presence of slickensides, brecciated rock, and possible weak zones will be cited under subheadings of the different concrete structures. Most all the concrete structures are founded on the competent dark gray to brown shale; a small portion of the lower approach wall is founded on the seamed limestone.

89. Lower approach wall. Possible weak zones in the bedrock beneath the lower approach wall include the discontinuous clay- and shale-filled partings in the limestone, the plastic clay layer in the green shale unit underlying the limestone, and the normal fault surface. The significance of the fault surfaces under the concrete structures will be discussed under subheading entitled "Tainter gate dam."

90. About 200 ft of the lower approach wall, downstream of the lower gate monoliths, are founded on the seamed limestone. The concrete and limestone were not bonded in one of the two borings made through the approach wall and into limestone. An angle of sliding friction (ϕ_r), $\phi_r = 30^{o}$, and cohesion (c), c = 0, is recommended for design values for that portion of the wall in contact with the limestone. Maximum and residual shear strengths of $\phi = 43^{o}$, c = 2.3 tsf and $\phi_r = 23.3^{o}$, c = 0 are recommended for design values of the clay-filled partings in limestone. Underlying the limestone is the green clay layer; residual shear values of $\phi_r = 12.5^{o}$ and c = 0 tsf are recommended for design consideration of this layer.

91. The remainder of the lower approach wall is assumed to be founded on the dark brown to gray shale. This material was in contact with the concrete in the farthest downstream boring. The contact was loose, and a $\phi_r = 20^{o}$ and c = 0 are recommended for design consideration.

92. Vertical and near vertical fractures were observed in the limestone; in general, the fracture surfaces were irregular and closed. Some fractures contained small amounts of clay, but the majority were free of deposits. The fracturing did not produce broken or blocky rock fragments and for the most part they were not interconnected. The extent or frequency of fracturing could not be determined from the borings

drilled in and adjacent to the wall. The vertical and near vertical fractures are assumed to parallel the northwest and northeast trending faults inferred at the site and are probably associated with the faulting. It is the author's opinion that the fractures would not control horizontal sliding beneath the approach wall.

93. Lock walls. The possible weak zones in the bedrock beneath the lock walls are the bedding planes in the dark brown to gray shale and the normal faults. The only boring in the lock was put through the land wall. Information from this one boring, from the structural contour map, cross sections, and evidences of sound engineer judgment used during construction of the lock and dam were used to assess the condition of the bedrock beneath the lock walls.

94. The boring in the land lock wall shows competent dark brown to gray shale underlying the concrete. There were no filled partings or other discontinuities in the core, and the concrete was well bonded to the bedrock. The concrete-rock contact is at El 466.0 ft, which is the founding elevation for the lock walls, as shown on the working drawings. The structural contour map and the cross section through the land lock wall (see Plates 12c and 13, respectively) indicate that with a founding elevation of 466.0 ft, most all portions of the lock walls are resting on the competent shale. The interpreted faulting illustrated in Plate 13 indicates that monoliths 24, 26, 28, 30, 32, and 34 are founded above the seamed limestone if these monoliths were founded at El 466.0 ft. However, it is reasonable to assume that the seamed limestone was removed during construction of the lock walls. This assumption is based on the actions taken by the engineers during construction of the two dam structures, as described in the following paragraph.

95. Borings logs show that the bases of the head gate and tainter gate dam sections were founded on the competent dark brown to gray shale. The base of the head gate dam was taken 18 ft below the base elevation shown on the working drawings. The base of the tainter gate dam was taken 7 to 9 ft below the base elevation as presented in the working drawings. The additional excavation was necessary to remove the seamed limestone in the dam area. None of the six borings through the dam

structures revealed any seamed limestone below concrete. Based on this evidence, it is assumed that both lock walls are founded on the competent shale. An angle of friction (ϕ) for the intact dark brown to gray shale, $\phi = 47.2^{o}$, and a cohesion of 3.6 tsf are recommended for design values in the foundation rock beneath the lock walls. Strength values for precut dark shale on dark shale of $\phi = 21.4^{o}$ and $c = 0.1$ tsf are recommended for lower bound cases.

96. Head gate dam. The possible weak zones in the bedrock beneath the head gate dam are the clay seams and normal faults. The competent dark brown to gray shale underlies the base of this structure. The contact between the concrete and the shale was well bonded in the two borings through the structure. Thin (about 1/2 in. thick) clay seams were detected in the core from 1 to 4 ft below the base. The extent of the seams is not known. Failure by sliding of head gate sections, on a clay seam, or shale bedding planes does not appear likely in view of the base elevation being some 18 ft beneath top of rock. Scour profiles indicate no significant scour depths downstream of the head gates. The green plastic clay layer underlying the Ft. Atkinson limestone is about 5 ft above the base of the head gate dam and just downstream of the dam; see Plate 19a. In order for this green clay layer to act as a horizontal sliding surface for the dam, it appears that a very large wedge of rock would have to be translated upward and downstream. Again, this appears unlikely in view of the mass of rock downstream of this structure. If failure within the limestone at the toe of the head gates occurred, cross-bed shear would likely be involved. The recommended shear strength values for cross-bed shear in the limestone are $\phi = 54.5^{o}$ and $c = 49.9$ tsf .

97. Tainter gate dam. The possible weak zones in the bedrock beneath the tainter gate dam are the shale bedding planes and the normal faults. The competent dark brown to gray shale underlies the base of the tainter gate dam. The contact between the concrete and the shale was well bonded in the four borings put through the structure. There

were no filled partings detected in the core within about 20 ft of the base. The design values for the intact dark brown to gray shale have been cited previously.

98. A study was made to see if any topographic feature downstream of the dam could be correlated with the interpreted faulting. At about 80 ft downstream of the apron and on scour profile 9R, scouring was found to be the deepest; see scour profile, gate 9R, on Plate A-51 in Appendix A. The bottom of the scouring in this area is at El 461.0 ft. Fault K intersects profile line 9R where the scour is at El 461.0 ft. The possibility that the scoured area is within the fault proper appears good. This is the only area so deeply scoured, and the area is some distance downstream of where the turbulent water is coming off the gate weirs. The other scour profile lines studied do not indicate any unusual topography other than trench features aligned normal to the dam. The trenches probably resulted from the water flow out of the gates. The deepest trenches are just downstream of the most frequently used gates.

99. Although no fault surface has been clearly identified during this investigation, numerous physical evidences of fault activity have been observed at the lock and dam site. There has not been any great amount of such evidence, except for the significant displacements of the marker bed over short distances. Brecciated rock in a clay matrix, vertical and near vertical fractures, slickensides on fractures in shale, and the above-mentioned marker bed displacements have been observed.

100. Photographs received from the ISGS show fault gouge in some of the fault zones they have trenched just south of the lock and dam. There is no way of knowing the number of faults that have gouge associated with them. The majority of faults at the lock and dam have relatively short throws, and the likelihood of the 16 faults interpreted at the site containing significant amounts of gouge appears small. If gouge or deposits do exist along the faults at the lock and dam, it seems that it would be well consolidated at this point in time. Movement in the foundation due to consolidation along the faults appears unlikely at this time. If the faults themselves are incoporated in a stability analysis, it is suggested that shear strength parameters presented for the green

plastic clay (c = 2.0 tsf and ϕ = 12.5^{o}) be considered. The clay would prescribe a lower shear resistance than the natural fractures would and would be a conservative estimate of the frictional properties along the faults if they did contain gouge or clay deposits.

101. As seen on the structural contour map, the faults are widely spaced (faults E, F, and G, H are about 100 ft apart) and in effect causing rather large blocks to exist. Of course, additional faults could occur between these blocks. It is difficult to imagine that the large blocks, interlocked as they are, could move horizontally even with the green plastic clay layers at the base of the blocks. In order for block movement to occur, an open face downstream of the dam would have to exist for a block to move. Scour profiles do not indicate that such an opening exists. The downstream area within 30 ft of the apron is scoured to a maximum depth of El 467 ft; the base elevation of the dam is 462.75. The base of the dam at the lowest scoured site is 4 ft below the scoured area. Plate 18 shows the green plastic clay at about base elevation of the dam. In order for a section of the dam to slide along the clay layer (contributing to failure), an opening would have to be created by a wedge of rock being translated upward and downstream.

102. The Sandwich Fault and those studied by the Survey adjacent to the lock and dam are reported to have been inactive for millions of years. Movement along the faults in the Dresden Island area during an earthquake seems unlikely since they have been inactive since Pennsylvanian time.

Concrete Condition

103. Boring L-8 in the land lock wall revealed 0.4 ft of new concrete, placed during a recent resurfacing operation, and 1.3 ft of highly deteriorated concrete beneath. The remaining 43 ft of concrete is in good condition. The top 0.4 ft of concrete in Boring D-9 in tainter gate pier No. 6 is highly deteriorated. The remaining 48.6 ft is in good condition. The interface between the concrete and the gray to brown

shale in these two borings is well bonded. Physical properties of the concrete were found to be quite similar to the physical properties reported in Reference 1.

Recommended Design Values

104. Design should consider rock type and the various bedrock structural features described herein. Guidance is presented in the following tabulation as to proper choice of design parameters. The tabulation is taken from Reference 1 and updated with values obtained during the compliance and scour detection studies.

	Limestone	Gry to Brn Shale	Grn Shale	Grn Clay
Characterization Properties				
Dry Unit Weight, lb/ft^3	175.0	144.8	155.8*	124.2
Effective Unit Weight, lb/ft^3	176.5	155.4	163.7*	138.6
Compressive Strength, psi	6280*	1830*	--	--
Tensile Strength, psi	--	195	--	--
Engineering Design Properties				
Shear Strength				
Intact	--	c=3.6 tsf* ϕ=47.2°	c=6.0 tsf* ϕ=38.3 tsf c=0 ϕ_r=34.3°	
Filled parting green clay seam	c=2.3 tsf* ϕ=43° c=0 ϕ_r=23.3°	--	--	c=0* ϕ=12.5° --
shale/clay seam	c=1.6 tsf* ϕ=34.3° c=0 ϕ_r=21°	--		
Precut, rock-on-rock	--	c=0 ϕ_r=21.4°	--	--
Concrete-on-rock	c=19 tsf ϕ=67.2° c=0 ϕ_r=37°	c=5.6 tsf ϕ=49°	--	--
Precut, concrete-on-rock	c=0 ϕ_r=30°**	c=0* ϕ_r=20°	--	--

	Limestone	Gry to Brn Shale	Grn Shale	Grn Clay
Cross-bed	c=49.9 tsf* ϕ=54.5^{o}	c=5.7 tsf ϕ=46.3^{o}	--	--
Modulus of Elasticity, 10^6 psi	5.51	0.25	--	--
Poisson's Ratio	0.26	0.36	--	--
Shear Modulus, 10^6 psi	2.19	0.10	--	--

* New data obtained during compliance and scour detection studies.
** Generally excepted values.

Recommended Future Work

105. If any part of the bedrock at the lock and dam site is exposed, such as by dewatering, it is recommended that detailed geologic mapping be performed. Such mapping and any additional boring information at the site could be useful in verifying or disproving the existence of faults.

106. No further drilling is recommended for investigating the faulting at the lock and dam site. The faults have been inactive for millions of years, and there appears to be no immediate evidence that settlement has or would occur along the faults.

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Table 1

Types, Locations, Elevations, and Starting Dates of Borings

Boring No.	Type of Boring	Location	El Top of Boring	El Top of Rock	El Bottom of Boring	Start Date
SACDI-1	▲	Backfill, lock	511.6	478.1	440.5	14 Jul 1971
SACDI-2	▲	Backfill, lock	510.1	479.5	429.8	18 Jul 1971
SACDI-4	▲	US of dam	492.3	476.8	448.1	26 Oct 1971
SACDI-6	▲	US of dam	502.1	479.0	450.6	27 Nov 1971
DI WES GW 2-77	◉	Lower guide wall	497.0	474.0	448.7	12 Apr 1977
DI WES GW 5-77	◉	Lower guide wall	497.0	472.6	449.0	15 Apr 1977
DI WES GW 8-77	●	Lower guide wall	497.0	474.5	463.5	16 May 1977
DI WES GWB 1-77	▲	Backfill, LGW	497.0	474.5	473.6	21 Apr 1977
DI WES GWB 2-77	▲̅	Backfill, LGW	510.2	496.4	475.0	25 Apr 1977
DI WES GWB 3-77	▲̅	Backfill, LGW	499.3	486.0	463.6	12 Apr 1977
DI WES MCA 1-77	◉	Mooring cell, US	511.0	460.0	433.8	9 Sep 1977
DI WES MCA 2-77	◉	Mooring cell, US	511.0	458.0	450.3	16 Sep 1977
DI WES MCB 3-77	◉	Mooring cell site, US	496.7	483.4	451.3	24 Sep 1977
DI WES MCB 4-77	◉	Mooring cell site, US	496.6	485.5	470.3	28 Sep 1977
DI WES MCB 5-77	◉	Mooring cell site, US	497.1	488.5	468.2	29 Sep 1977
DI WES MCB 6-77	◉	Mooring cell site, US	495.2	486.8	467.6	1 Oct 1977
DI WES MCB 7-77	◉	Mooring cell site, US	495.0	486.7	468.0	5 Oct 1977
DI WES D 9-77	◉	Tainter gate pier	511.8	462.8	440.9	15 Aug 1977
DI WES E 1-77	▲̅	RH abutment (E)	512.0	477.8	461.0	26 Aug 1977
DI WES E 2-77	▲̅	RH abutment (E)	511.2	477.3	460.6	1 Sep 1977
DI WES L 8-77	◉	Land lock wall	510.0	466.4	429.2	20 Aug 1977
DI WES D 29-78	◉	Dam, DS	473.0	473.0	462.8	8 Jul 1978
DI WES D 30-78	◉	Dam, DS	474.5	474.5	464.5	9 Jul 1978
DI WES D 31-78	◉	Dam, DS	473.2	473.2	463.6	11 Jul 1978
DI WES D 32-78	◉	Dam, DS	476.2	476.2	466.6	7 Jul 1978
DI WES D 33-78	◉	Dam, DS	474.0	474.0	463.9	12 Jul 1978
DI WES D 34-78	◉	Dam, DS	470.9	470.9	462.2	6 Jul 1978
DI WES D 35-78	◉	Dam, DS	474.0	474.0	459.9	13 Jul 1978
DI WES D 36-78	◉	Dam, DS	472.9	472.9	437.1	18 Jul 1978
DI WES D 37-78	◉	Dam, DS	478.9	472.4	467.1	6 Jul 1978
DI WES D 38-78	◉	Dam, DS	473.9	473.9	464.2	14 Jul 1978
DI WES D 39-78	◉	Dam, DS	475.4	475.4	466.8	15 Jul 1978
DI WES D 40-78	◉	Dam, DS	471.6	471.6	462.4	17 Jul 1978
DI WES D 41-78	◉	Dam, DS	478.5	477.4	468.8	10 Jul 1978
DI WES D 42-78	◉	Dam, DS	474.3	474.3	465.8	20 Jul 1978
DI WES D 43-78	◉	Dam, DS	482.1	476.8	439.5	21 Jul 1978
DI WES D 44-78	◉	Dam, DS	473.5	473.5	464.9	25 Jul 1978
DI WES D 45-79		Head gate	513.6	459.9	454.1	8 Mar 1979
DI WES D 46-79		Head gate	513.6	459.9	453.7	6 Mar 1979
DI WES D 47-79		Tainter gate bay	479.7	462.4	436.2	10 Apr 1979
DI WES D 48-79		Tainter gate bay	479.5	462.7	437.4	14 Apr 1979
DI WES D 49-79		Tainter gate bay	479.9	465.1	437.4	5 Apr 1979

▲̅ Combined drive sample and core with piezometer installed.

▲ Combined drive sample and core; SACDI borings with 4-in. core recovered and WES borings with 6-in. core recovered.

● 3-in. core hole.

◉ 6-in. core hole.

US = upstream, DS = downstream, RH abutment (E) = right-hand abutment (embankment), LGW = lower guide wall.

Table 2

WES Core from Dresden Island Lock and Dam, Illinois Waterway,

Chicago District (Compliance Phase)

WES Reference	Drill Hole No.	Date Rec'd	Core Diam in.	Bag or Box No.	Depth, ft	Elevation, ft Depth Intervals	Elevation, ft Top of Hole	Remarks
CHI 12 DC 41 (A)	DI WES D 9-77	9-30-77	6	1 of 17	0.0-3.3	511.75-508.45	511.75	Concrete
(B)				2	3.3-7.0	508.45-504.75		
(C)				3	7.0-9.6	504.75-502.15		
(C)				3	14.5-16.1	497.25-495.65		
(D)				4	9.6-14.5	502.15-497.25		
(E)				5	16.1-21.0	495.65-490.75		
(E)				6	21.0-25.8	490.75-485.95		
(G)				7	25.8-30.1	485.95-481.65		
(H)				8	30.1-32.9	481.65-478.85		
(H)				8	36.5-38.5	475.25-473.25		
(I)				9	32.9-36.5	478.85-475.25		
(I)				9	36.5-39.5	475.25-472.25		
(J)				10	39.5-42.5	472.25-469.25		Concrete
(J)				10	49.1-50.0	462.65-461.75		Shale
(K)				11	42.5-46.2	469.25-465.55		Concrete
(K)				11	50.0-51.1	461.75-460.65		Shale
(L)				12	46.2-49.1	465.55-462.65		Concrete
(L)				12	51.1-52.6	460.65-459.15		Shale
(M)				13	52.6-56.6	459.15-455.15		
(N)				14	56.6-60.0	455.15-451.75		
(O)				15	60.0-64.5	451.75-447.25		
(P)				16	64.5-68.7	447.25-443.05		
CHI 12 DC 41 (Q)	DI WES D 9-77		6	17 of 17	68.7-70.9	443.05-440.85		Shale
CHI 12 DC 42 (A)	DI WES L 8-77		6	1 of 19	0.0-3.1	510.0-506.9	510.0	Concrete
(A)				1	4.8-5.6	505.2-504.4		
(B)				2	3.1-4.8	506.9-505.2		
(B)				2	5.6-7.3	504.4-502.7		
(C)				3	7.3-9.9	502.7-500.1		
(C)				3	19.5-21.9	490.5-488.1		
(D)				4	9.9-14.9	500.1-495.1		
(E)				5	14.9-19.5	495.1-490.5		
(F)				6	21.9-26.6	488.1-483.4		
(G)				7	26.6-31.2	483.4-478.8		
(H)				8	31.2-34.9	478.8-475.1		
(I)				9	34.9-37.5	475.1-472.5		
(I)				9	41.7-43.2	468.3-466.8		
(J)				10	37.5-41.7	472.5-468.3		Concrete
(K)				11	43.2-47.6	466.8-462.4		Concrete, shale
(L)				12	47.6-51.6	462.4-458.4		Shale
(M)				13	51.6-55.9	458.4-454.1		
(N)				14	55.9-60.6	454.1-449.4		
(O)				15	60.6-65.2	449.4-444.8		
(P)				16	65.2-69.9	444.8-440.1		
(Q)				17	69.9-74.4	440.1-435.6		
(R)				18	74.4-78.9	435.6-431.1		
CHI 12 DC 42 (S)	DI WES L 8-77		6	19 of 19	78.9-80.8	431.1-429.2		Shale
CHI 12 DC 43 (A)	DI WES E 1-77		6	1 of 8	2.6-8.0	509.4-504.0	512.0	Gravel, sand, clay
(B)				2	8.0-13.2	504.0-498.8		
(C)				3	13.2-16.6	498.8-495.4		
(D)				4	16.6-25.3	495.4-486.7		
(E)				1 Bag	25.3-34.2	486.7-477.8		Gravel, sand, clay
(F)				5	34.2-38.4	477.8-473.6		Limestone
(G)				6	38.4-43.3	473.6-468.7		Limestone
(H)				7	43.3-47.1	468.7-464.9		Limestone, shaley
CHI 12 DC 43 (I)	DI WES E 1-77		6	8 of 8	47.1-51.0	464.9-461.0		Limestone, shale
CHI 12 DC 44 (A)	DI WES E 2-77		6	1 Bag	0.0-8.8	511.2-502.4	511.2	Gravel, sand, clay
(B)				1 of 5	8.8-14.8	502.4-496.4		
(C)				2	14.8-19.3	496.4-491.9		
(D)				2 Bag	19.3-20.5	491.9-490.7		
(F)				3	20.5-24.8	490.7-486.4		
(F)			6	3 Bag	24.8-34.3	486.4-476.9		Gravel, sand, clay, limestone
(G)			2.125	4	34.3-48.6	476.9-462.6		Limestone, shaley
CHI 12 DC 44 (H)	DI WES E 2-77		2.125	5 of 5	48.6-50.6	462.6-460.6		Limestone, clayey shale

Table 3

WES Cores from Dresden Island Lock and Dam, Illinois Waterways,

Chicago District (Sonic Detection)

WES Reference	Drill Hole No.	Date Rec'd	Core Diam in.	Box No.	Depth, ft	Elevation, ft Depth Intervals	Top of Hole	Remarks
CHI 12 DC 45 (A)	DI WES D 29-78	7-31-78	6	1 of 2	0.0-4.4	473.0-468.6	473.0	
(B)				2 of 2	4.4-8.9	468.6-464.1		
(C)	DI WES D 29-78	7-3178	6	2 of 2	8.9-10.25	464.1-462.75		
CHI 12 DC 46 (A)	DI WES D 30-78	7-31-78	6	1 of 2	0.0-3.9	474.5-470.6	474.5	
(B)				2 of 2	3.9-8.8	470.6-465.7		
(C)	DI WES D 30-78	7-31-78	6	2 of 2	8.8-9.8	465.7-464.7		
CHI 12 DC 47 (A)	DI WES D 31-78	7-31-78	6	1 of 2	0.0-4.2	473.2-469.0	473.2	
(B)				2 of 2	4.2-8.3	469.0-464.9		
(C)	DI WES D 31-78	7-31-78	6	2 of 2	8.3-9.53	464.9-463.67		
CHI 12 DC 48 (A)	DI WES D 32-78	7-31-78	6	1 of 2	0.0-4.3	476.2-471.9	476.2	
(B)				2 of 2	4.3-8.1	471.9-468.1		
(C)	DI WES D 32-78	7-31-78	6	2 of 2	8.1-9.85	468.1-466.35		
CHI 12 DC 49 (A)	DI WES D 33-78	7-31-78	6	1 of 2	0.0-4.4	474.0-469.5	474.0	
(B)				2 of 2	4.4-8.85	469.5-465.15		
(C)	DI WES D 33-78	7-31-78	6	2 of 2	8.85-10.1	465.15-463.9		
CHI 12 DC 50 (A)	DI WES D 34-78	7-31-78	6	1 of 2	0.0-4.2	470.4-466.2	470.4	
(B)	DI WES D 34-78	7-31-78	6	2 of 2	4.2-7.8	466.2-462.6		
CHI 12 DC 51 (A)	DI WES D 35-78	7-31-78	6	1 of 3	0.0-4.3	474.0-469.7	474.0	
(B)				2 of 3	4.3-9.0	469.7-465.0		
(C)	DI WES D 35-78	7-31-78	6	3 of 3	9.0-14.05	465.0-459.95		
CHI 12 DC 52 (A)	DI WES D 36-78	7-31-78	6	1 of 9	0.0-4.5	472.9-468.4	472.9	
(B)				2 of 9	4.5-8.1	468.4-464.8		
(C)				3 of 9	8.1-12.3	464.8-460.6		
(D)				4 of 9	12.3-16.6	460.6-456.3		
(E)				5 of 9	16.6-20.8	456.3-452.1		
(F)				6 of 9	20.8-24.3	452.1-448.6		
(G)				7 of 9	24.3-29.1	448.6-443.8		
(H)				8 of 9	29.1-32.6	443.8-440.3		
(I)	DI WES D 36-78	7-31-78	6	9 of 9	32.6-35.8	440.3-437.1		
CHI 12 DC 53 (A)	DI WES D 37-78	7-31-78	6	1 of 2	0.0-8.4	478.9-470.5	478.9	
(B)	DI WES D 37-78	7-31-78	6	2 of 2	8.4-11.8	470.5-467.1		
CHI 12 DC 54 (A)	DI WES D 38-78	7-31-78	6	1 of 2	0.0-4.7	473.9-469.2	473.9	
(B)				2 of 2	4.7-8.8	469.2-465.1		
(C)	DI WES D 38-78	7-31-78	6	2 of 2	8.8-9.3	465.1-464.6		
CHI 12 DC 55 (A)	DI WES D 39-78	7-31-78	6	1 of 2	0.0-4.5	475.4-470.9	475.4	
(B)	DI WES D 39-78	7-31-78	6	2 of 2	4.5-8.6	470.9-466.8		
CHI 12 DC 56 (A)	DI WES D 40-78	7-31-78	6	1 of 2	0.0-4.6	471.6-467.0	471.6	
(B)	DI WES D 40-78	7-31-78	6	2 of 2	4.6-9.2	467.0-462.4		
CHI 12 DC 57 (A)	DI WES D 41-78	7-31-78	6	1 of 2	0.0-4.2	478.5-474.3	478.5	
(B)				2 of 2	4.2-8.5	474.3-470.0		
(C)	DI WES D 41-78	7-31-78	6	2 of 2	8.5-9.75	470.0-468.75		
CHI 12 DC 58 (A)	DI WES D 42-78	7-31-78	6	1 of 2	0.0-3.8	474.3-470.5	474.3	
(B)	DI WES D 42-78	7-31-78	6	2 of 2	3.8-8.5	470.5-465.8		
CHI 12 DC 59 (A)	DI WES D 43-78	7-31-78	6	1 of 10	0.0-6.5	482.08-475.58	482.08	
(B)				2 of 10	6.5-10.4	475.58-471.68		
(C)				4 of 10	14.3-16.45	467.78-465.63		
(D)				5 of 10	16.7-21.1	465.38-460.98		
(E)				6 of 10	21.1-25.7	460.98-456.38		
(F)				7 of 10	25.7-29.9	456.38-452.18		
(G)				8 of 10	29.9-33.35	452.18-448.73		
(H)				9 of 10	33.35-37.5	448.73-444.58		
(I)	DI WES D 43-78	7-31-78	6	10 of 10	37.5-41.4	444.58-440.68		
CHI 12 DC 60 (A)	DI WES D 44-78	7-31-78	6	1 of 2	0.0-4.5	473.4-468.9	473.4	
(B)	DI WES D 44-78	7-31-78	6	2 of 2	4.5-8.5	468.9-464.9		

Table 4

Characterization and Engineering Design Test Results of Concrete and Shale and Limestone, Compliance Phase and Scour Detection, Dresden Island Lock and Dam

Drill Hole No. DI WES	Elevation ft	Characterization Tests: Effective Unit Weight γ_m lb/ft^3	Effective Unit Weight γ_d lb/ft^3	Water Content w, %	Compressive Wave Velocity V_p ft/sec	Compressive Strength UC, psi	Engineering Design Tests: Elastic Modulus $\times 10^6$ psi	Poissons Ratio	Rock Type
Concrete Test Results Compliance Phase									
D 9-77	509.1	153.0	140.9	8.6	14,925	5,830	2.50	0.28	
D 9-77	491.3	151.1	141.6	6.7	16,005	6,040	5.00	0.22	
D 9-77	463.8	152.3	142.6	6.8	15,625	5,700	4.57	0.21	
L 8-77	506.4	151.1	142.5	6.0	15,623	6,570	4.65	0.38	
L 8-77	484.5	151.1	141.0	7.2	15,512	7,230	4.49	0.20	
L 8-77	468.8	149.2	133.0	8.1	15,625	5,740	4.30	0.22	
Average		151.3	141.1	7.2	15,553	6,190	4.25	0.25	
Range		3.8	4.6	2.6	1,080	1,490	2.50	0.18	
No. of tests		6	6	6	6	6	6	6	
Shale and Limestone Test Results, Compliance Phase and Scour Detection									
D 9-77	459.9	158.6	151.6	4.6	9,350	2,880	0.63	0.17	Gray to brown shale
D 9-77	456.8	159.2	150.9	5.5	9,925	2,620	0.86	0.17	Gray to brown shale
D 9-77	455.8	154.2	145.3	6.1	8,095	1,830	0.58	0.26	Gray to brown shale
E 1-77	467.7	165.4	161.4	2.5	17,575	2,380	--	--	Limestone, vuggy, clayey
E 1-77	465.7	173.6	171.4	1.3	20,460	12,990	--	--	Limestone
E 1-77	464.3	171.1	166.9	2.5	16,390	5,850	--	--	Limestone
E 1-77	462.7	172.9	169.5	2.0	17,545	7,360	--	--	Limestone
D 32-78	475.7	--	--	--	--	5,430	3.3	--	Limestone, vuggy, and fossiliferous
D 32-78	473.7	--	--	--	--	5,480	3.4	--	Limestone, vuggy, and fossiliferous
D 36-78	470.7	--	--	--	--	4,490	2.9	--	Limestone, vuggy, and fossiliferous
Gray to brown shale									
Average		157.3	149.3	5.4	9,123	2,440	0.69	0.20	
Range		5.0	6.3	1.5	1,830	1,050	0.28	0.09	
No. of tests		3	3	3	3	3	3	3	
Limestone									
Average		170.8	167.3	2.1	17,992	6,280	3.20	--	
Range		8.2	10.0	1.2	4,070	10,610	0.50	--	
No. of tests		4	4	4	4	7	3	--	

Table 5

Engineering Design Test Results, Embankment,

Dresden Island Lock and Dam

Drill Hole No. DI WES	Elevation ft	Triaxial Total Stress ϕ	Triaxial Total Stress c, tsf	Triaxial Effective Stress ϕ'	Triaxial Effective Stress c', tsf
		Triaxial, $\bar{R}$ Test			
E 1-77	494.6-495.0	16.5°	0.3	37°	0.0

Drill Hole No. DI WES	Elevation ft	ϕ'	c', tsf
	Direct Shear		
E 1-77	496.0-496.4	31°	0.11
E 1-77	494.1-494.6	32°	0.0

PIEZOMETER INSTALLATION REPORT

PROJECT: Dresden Island Dam
LEVEE DISTRICT: Chicago District

LOCATION (STA.): See Note (1)
OFFSET FROM CENTER LINE: -
PIEZ NO: -

PIEZ TYPE: Silica Sand Bonded to Slotted $1\frac{1}{4}$" PVC Pipe (2)
DEPTH OF PIEZ: 26.3'
RISER PIPE DIAM: $1\frac{1}{2}$"

PIEZ TIP SET IN (SOIL TYPE): Clayey Gravel
SOIL SAMPLE NO: -
BORING DIAM: 7-3/4"

METHOD OF INSTALLATION: Piezometer set in a open hole.

TYPE OF PROTECTION FOR PIEZ: Riser encased in concrete from Elev. 498.2 to the ground surface.
VENT: 1/8" dia. hole in cap.

GROUND ELEV: 512.0 (3)
ELEV TOP OF RISER: 513.5
ELEV PIEZ TIP: 485.7 (4)

FILTER: Concrete Sand
FROM ELEV: 484.2
TO ELEV: 488.2

SEAL: Bentonite-cement-water grout
FROM ELEV: 488.2
TO ELEV: 498.2

INSTALLED BY: WES
CONTRACT NO: -
FOREMAN: McGee

DATE OF INSTALLATION: 1 Sept. 1977
DATE OF OBSERVATIONS: (5)

METHOD OF TESTING PIEZ: (5)

TIME	ELAPSED TIME MINUTES	DEPTH TO WATER FEET	TIME	ELAPSED TIME MINUTES	DEPTH TO WATER FEET	TIME	ELAPSED TIME MINUTES	DEPTH TO WATER FEET

REMARKS: (1) Piez. set in Boring DI-WES-E-1-77.

(2) O.D. of bonded silica tip is 3.0", length is 1.0'.

(3) Elevation determined with a hand level.

(4) Elevation of tip bottom.

(5) No observations or tests run on piez.

Mark A. Vispi for Robert Neal
INSPECTOR

WES FORM 793
MAR. 53
REVISED OCT '53

PLATE 1

PIEZOMETER INSTALLATION REPORT

PROJECT: Dresden Island Dam

LEVEE DISTRICT: Chicago District

LOCATION (STA.): See Note (1)

OFFSET FROM CENTER LINE: -

PIEZ NO: -

PIEZ TYPE: Silica Sand Bonded to Slotted $1\frac{1}{4}$" PVC Pipe (2)

DEPTH OF PIEZ: 32.8'

RISER PIPE DIAM: $1\frac{1}{2}$"

PIEZ TIP SET IN (SOIL TYPE): Gravelly Sand

SOIL SAMPLE NO: -

BORING DIAM: 7-3/4"

METHOD OF INSTALLATION: Piezometer set in a open hole.

TYPE OF PROTECTION FOR PIEZ: Riser encased in concrete from Elev. 495.9 to the ground surface.

VENT: 1/8" dia. hole in cap

GROUND ELEV: 511.2 (3)

ELEV TOP OF RISER: 512.7

ELEV PIEZ TIP: 478.4 (4)

FILTER: Concrete Sand FROM ELEV: 476.9 TO ELEV: 480.9

SEAL: Bentonite-cement-water grout FROM ELEV: 480.9 TO ELEV: 495.9

INSTALLED BY: WES

CONTRACT NO: -

FOREMAN: McGee

DATE OF INSTALLATION: 6 Sept. 1977

DATE OF OBSERVATIONS: (5)

METHOD OF TESTING PIEZ: (5)

TIME	ELAPSED TIME MINUTES	DEPTH TO WATER FEET	TIME	ELAPSED TIME MINUTES	DEPTH TO WATER FEET	TIME	ELAPSED TIME MINUTES	DEPTH TO WATER FEET

REMARKS: (1) Piez. set in Boring DI-WES-E-2-77.

(2) O.D. of bonded silica tip is 3.0", length is 1.0'.

(3) Elevation determined with a hand level.

(4) Elevation of tip bottom.

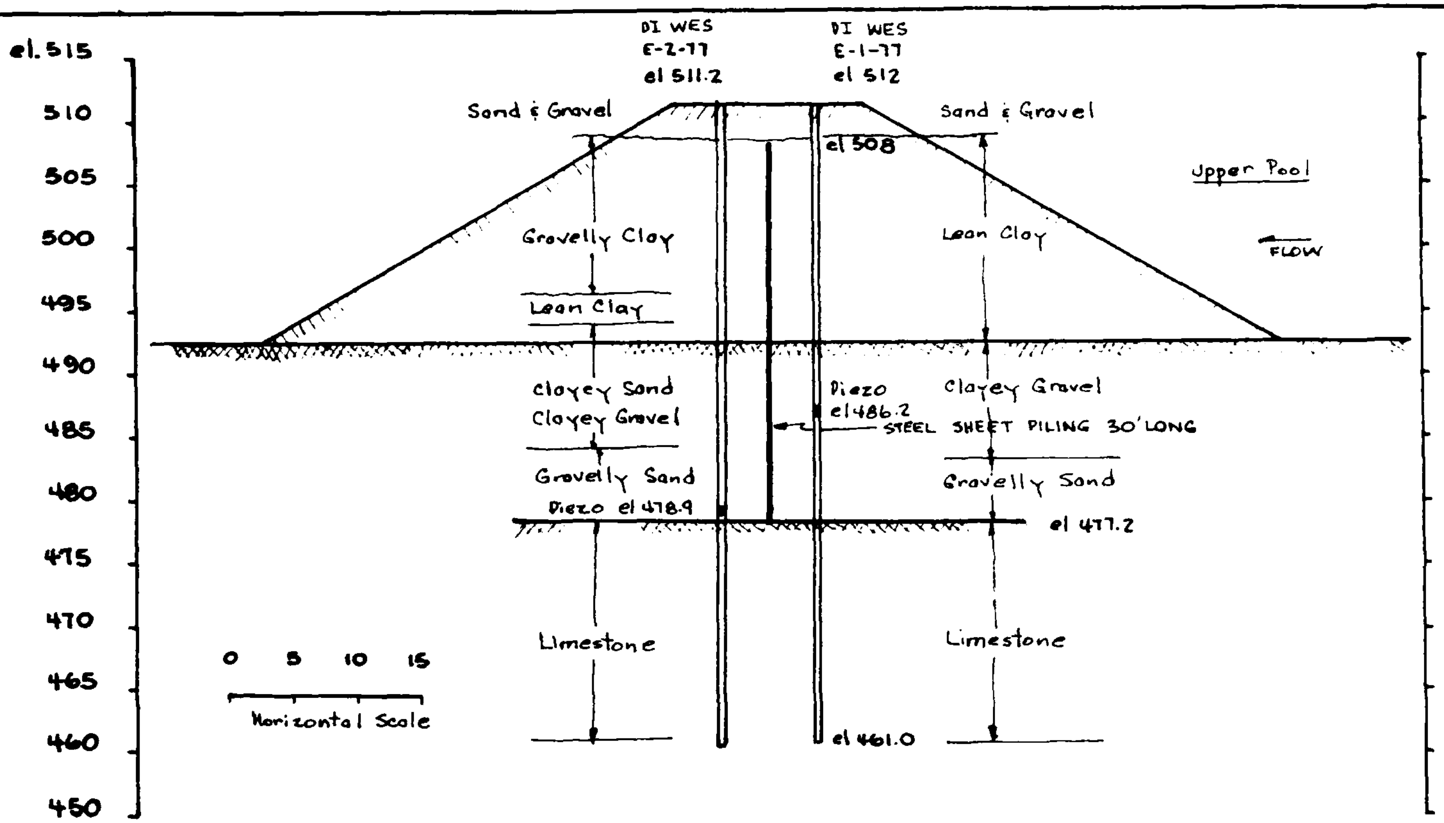
(5) No observations or tests run on piez.

Mark A. Vispi for Robert Neal

INSPECTOR

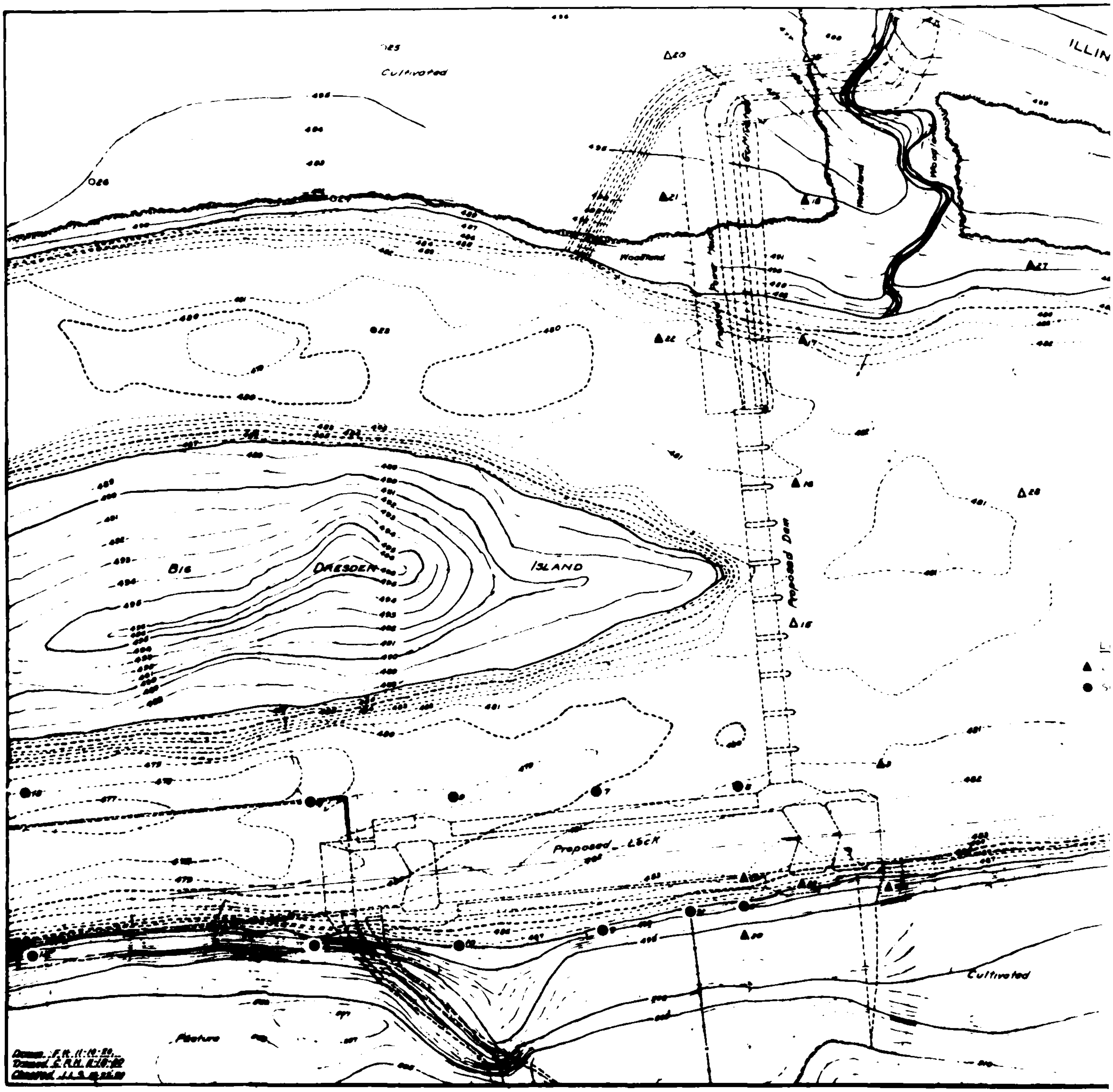
WES FORM 793
MAR. 53
REVISED OCT '53

PLATE 2

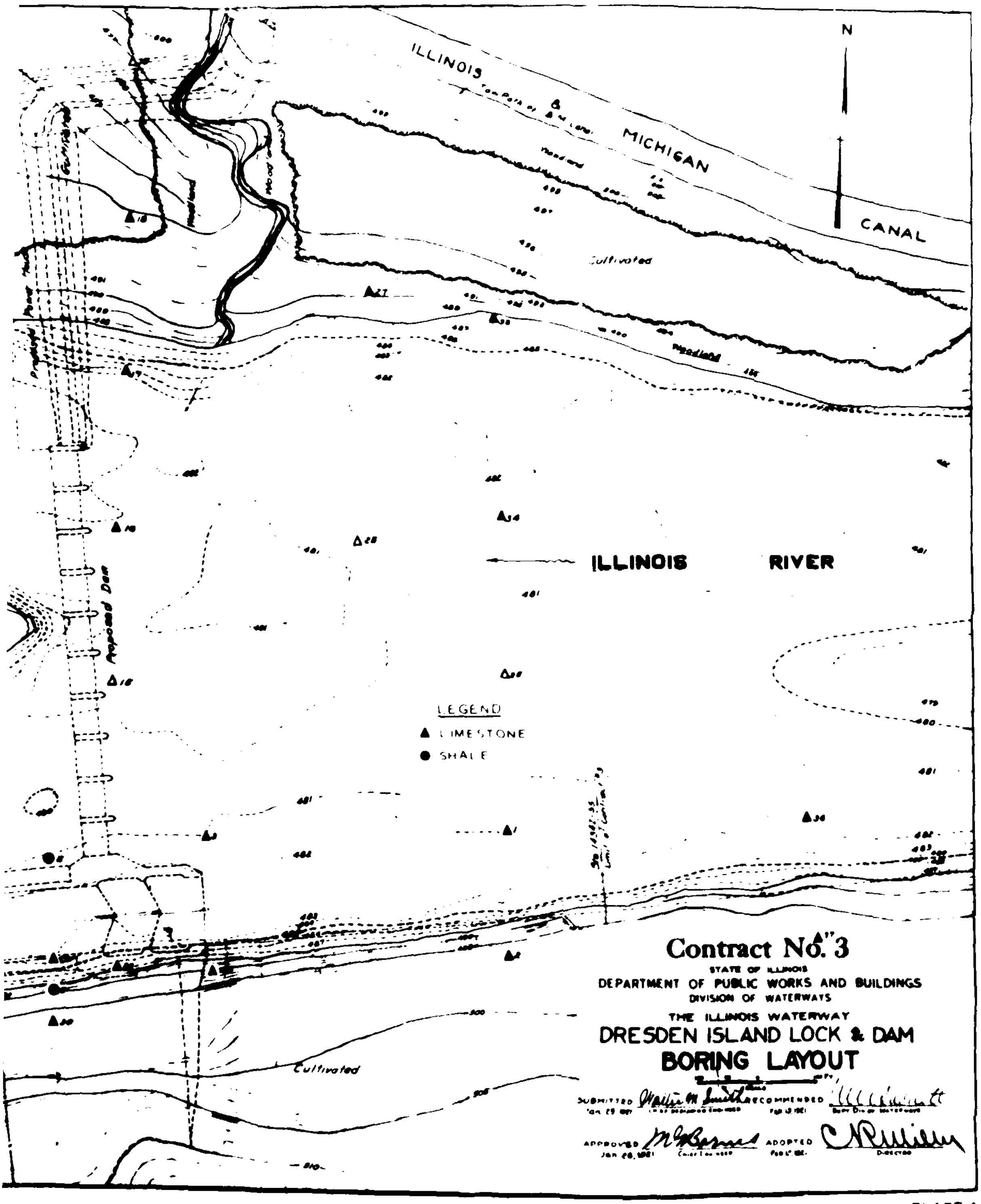


Boring No.	El Top Hole, Ft	Date Complete	Date of Reading	Water El, Ft	Upper Pool El, Ft	Lower Pool El, Ft
DI WES E-1-77	512	9-1-77	9-2-77	500.0	505.2	491.8
			9-3-77	500.5	505.2	488.0
			9-5-77	500.1	505.2	486.8
			9-6-77	500.6	505.2	486.5
			9-19-77	498.4	--	--
DI WES E-2-77	511.2	9-6-77	9-19-77	491.8	--	--

Section of Embankment at F-F', Dresden Island Lock and Dam

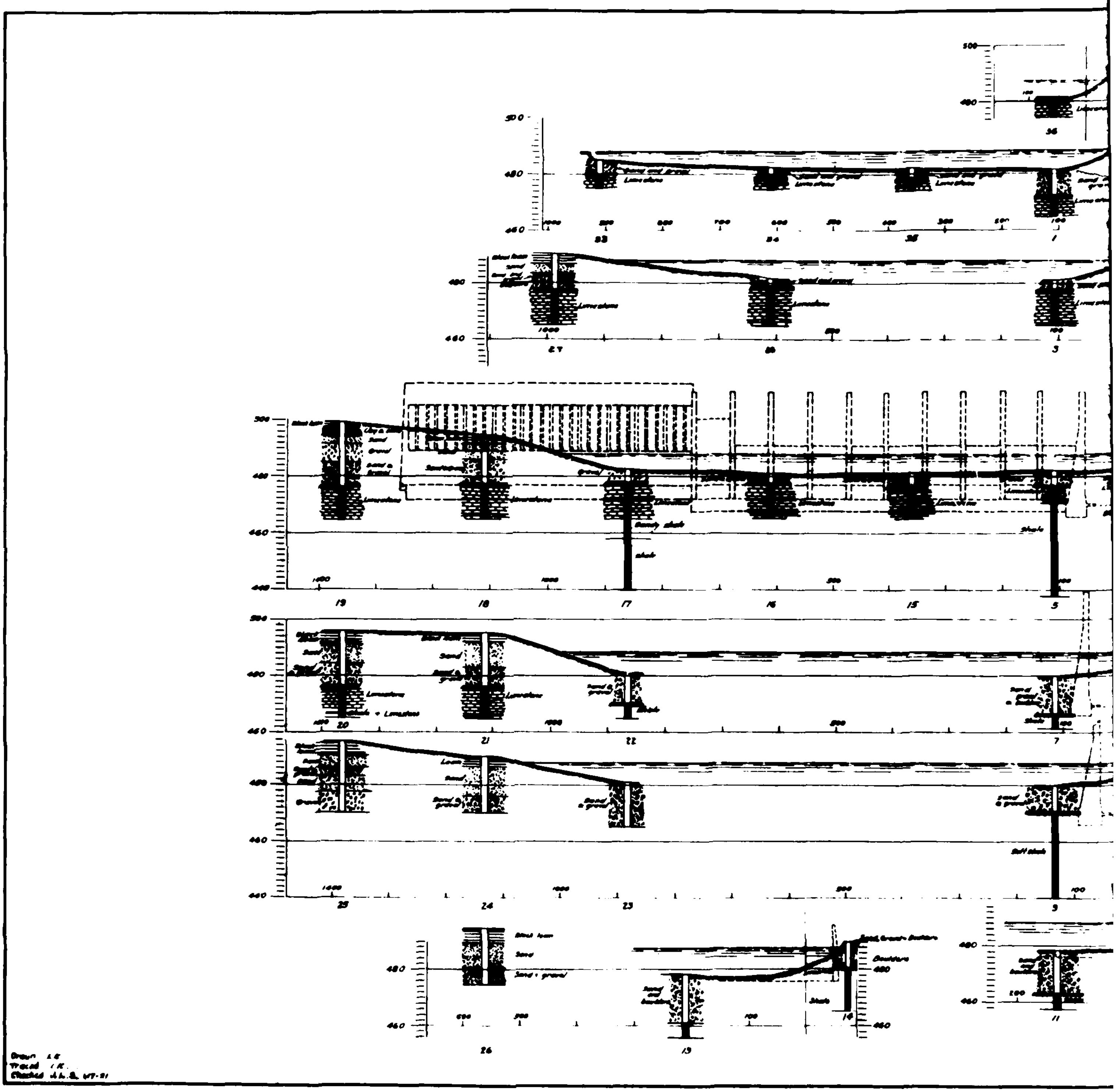

Cultivated
ILLIN
Woodland
Proposed Dam
BIG
DRESDEN
ISLAND
Proposed LOCK
Cultivated
Pasture

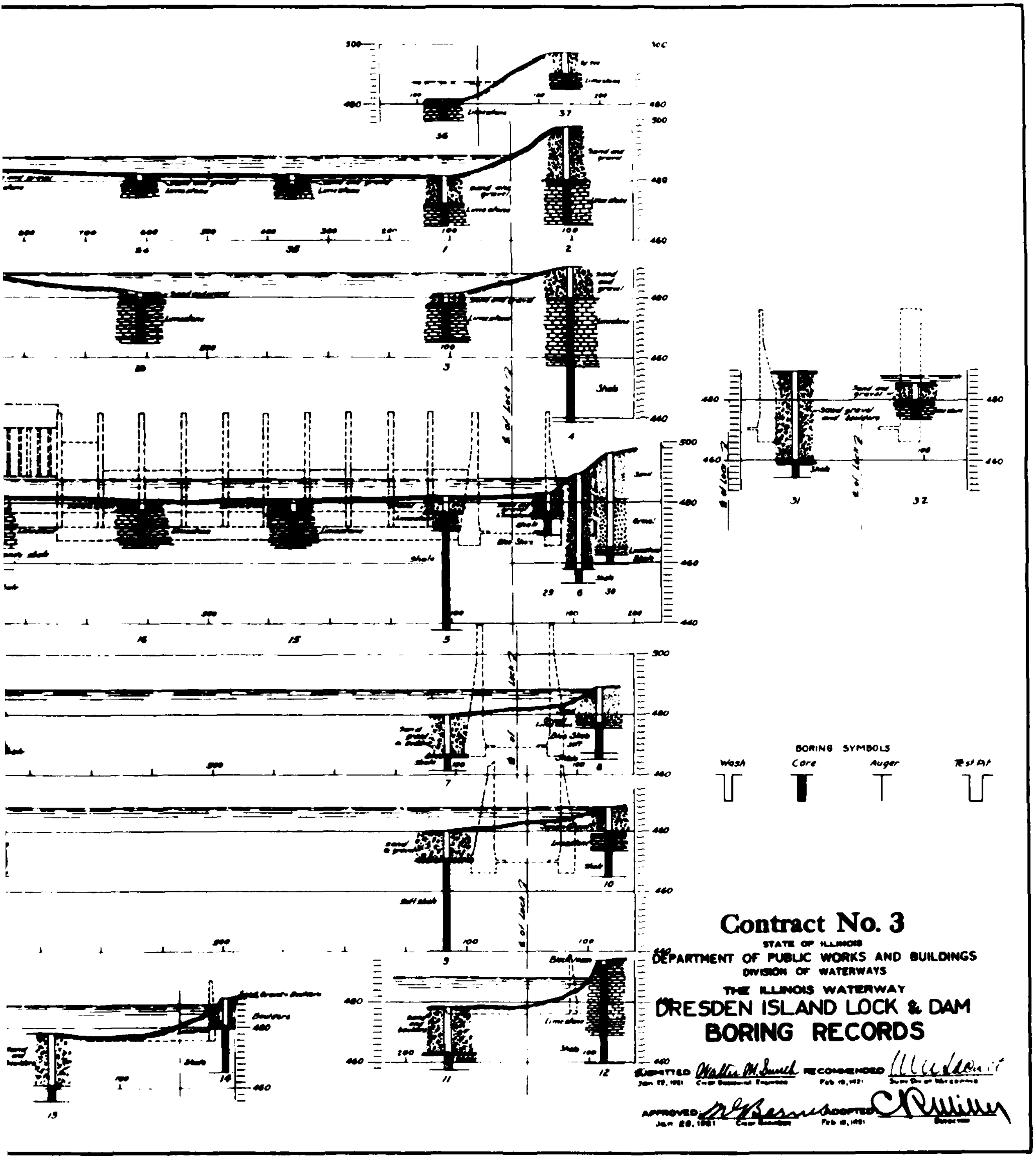
1



ILLINOIS & MICHIGAN CANAL
N
Woodland
Cultivated
ILLINOIS RIVER
Proposed Dam
LEGEND
LIMESTONE
SHALE
Contract No. A 3
STATE OF ILLINOIS
DEPARTMENT OF PUBLIC WORKS AND BUILDINGS
DIVISION OF WATERWAYS
THE ILLINOIS WATERWAY
DRESDEN ISLAND LOCK & DAM
BORING LAYOUT

PLATE 4


500
480
460
440
Limestone
Sand and gravel
Shale
Soft shale
Black loam
Sand
Sand + gravel
Boulders



Contract No. 3

STATE OF ILLINOIS
DEPARTMENT OF PUBLIC WORKS AND BUILDINGS
DIVISION OF WATERWAYS

THE ILLINOIS WATERWAY

DRESDEN ISLAND LOCK & DAM
BORING RECORDS

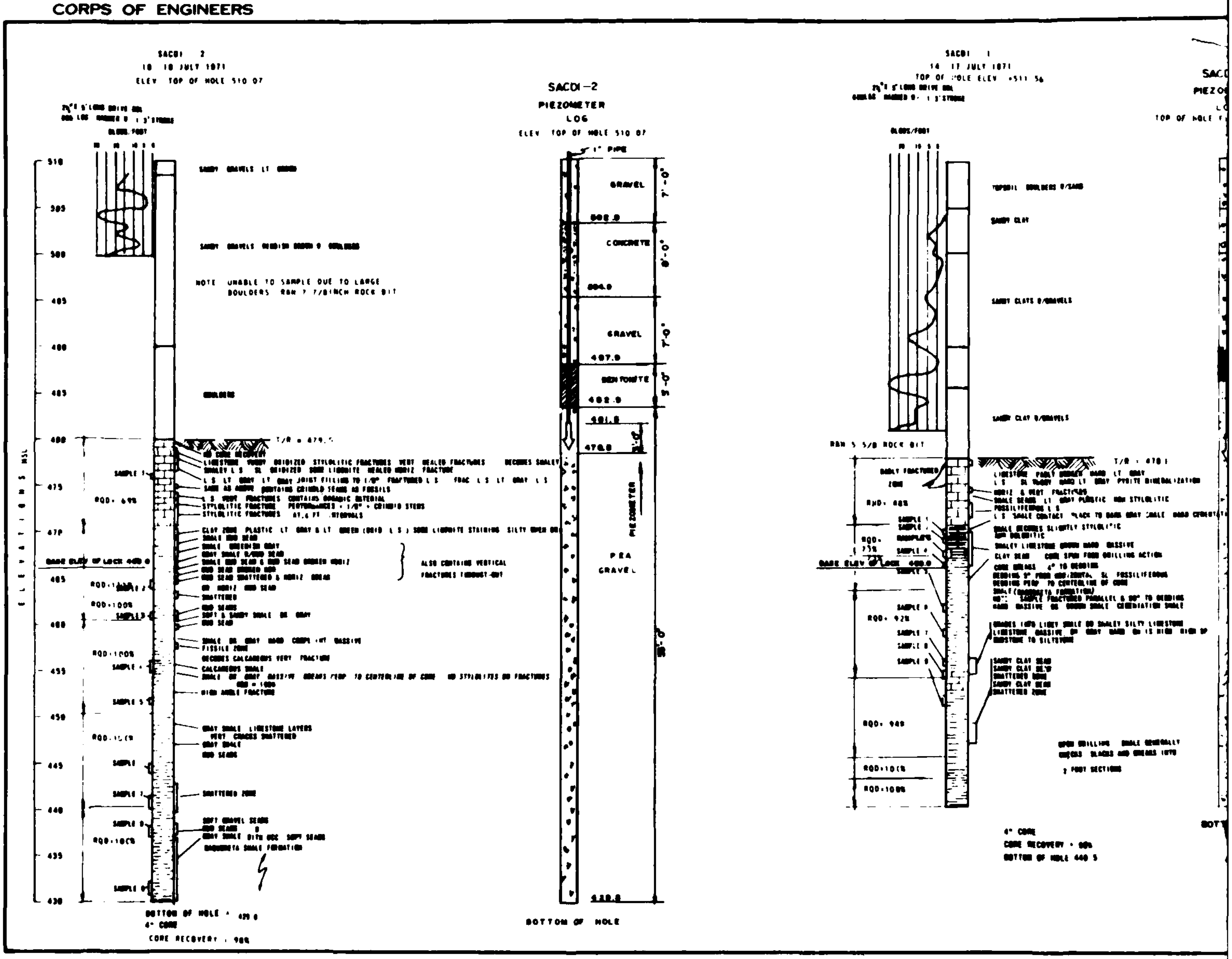
SUBMITTED — Jan 29, 1931 — RECOMMENDED — Feb 10, 1931

APPROVED — Jan 29, 1931 — ADOPTED — Feb 10, 1931

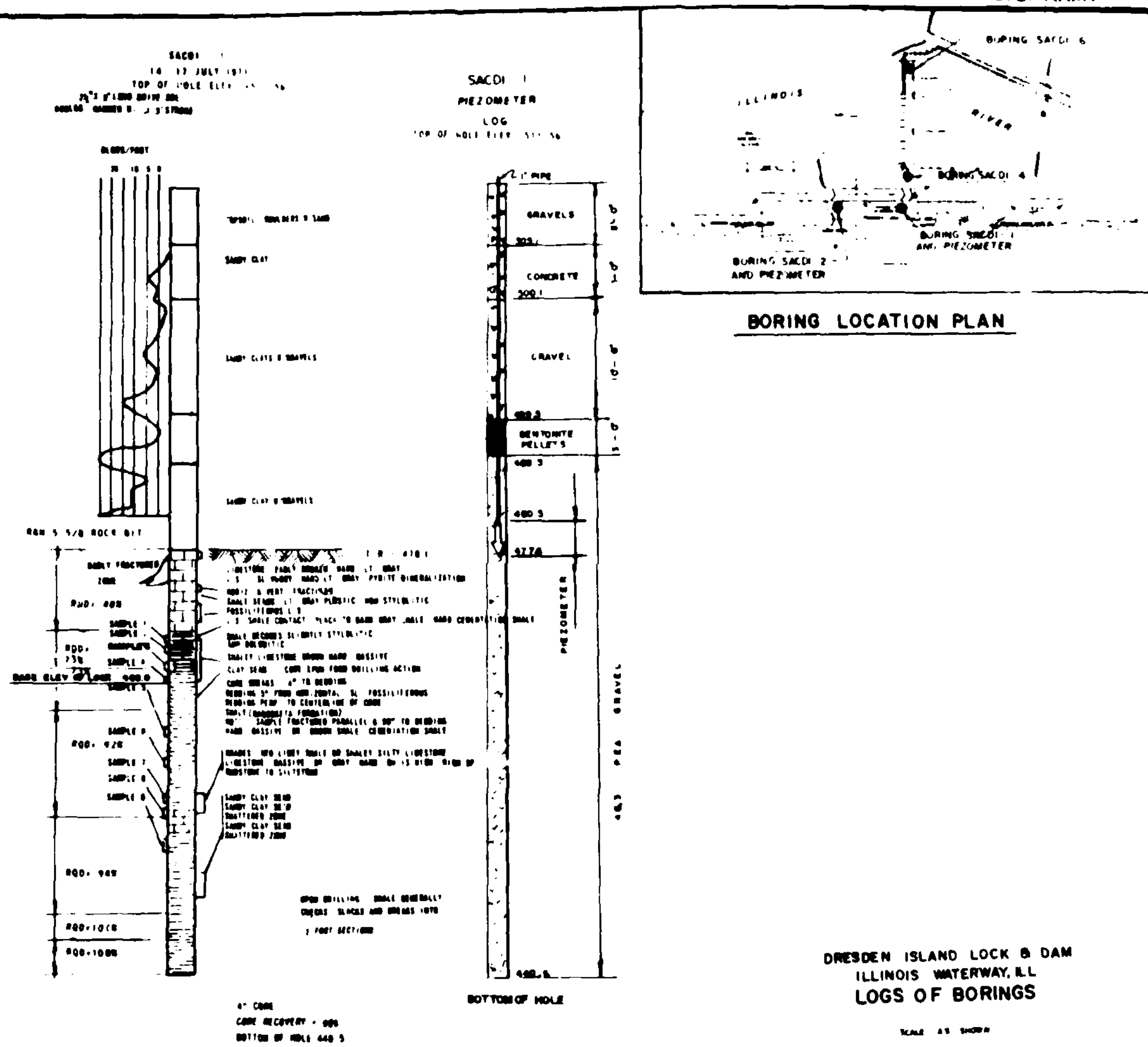
PLATE 5

2

CORPS OF ENGINEERS



U.S. ARMY



BORING LOCATION PLAN

DRESDEN ISLAND LOCK & DAM
ILLINOIS WATERWAY, ILL
LOGS OF BORINGS

SCALE AS SHOWN

DEPARTMENT OF THE ARMY CHICAGO DISTRICT

CORPS OF ENGINEERS ENGINEERING DIVISION

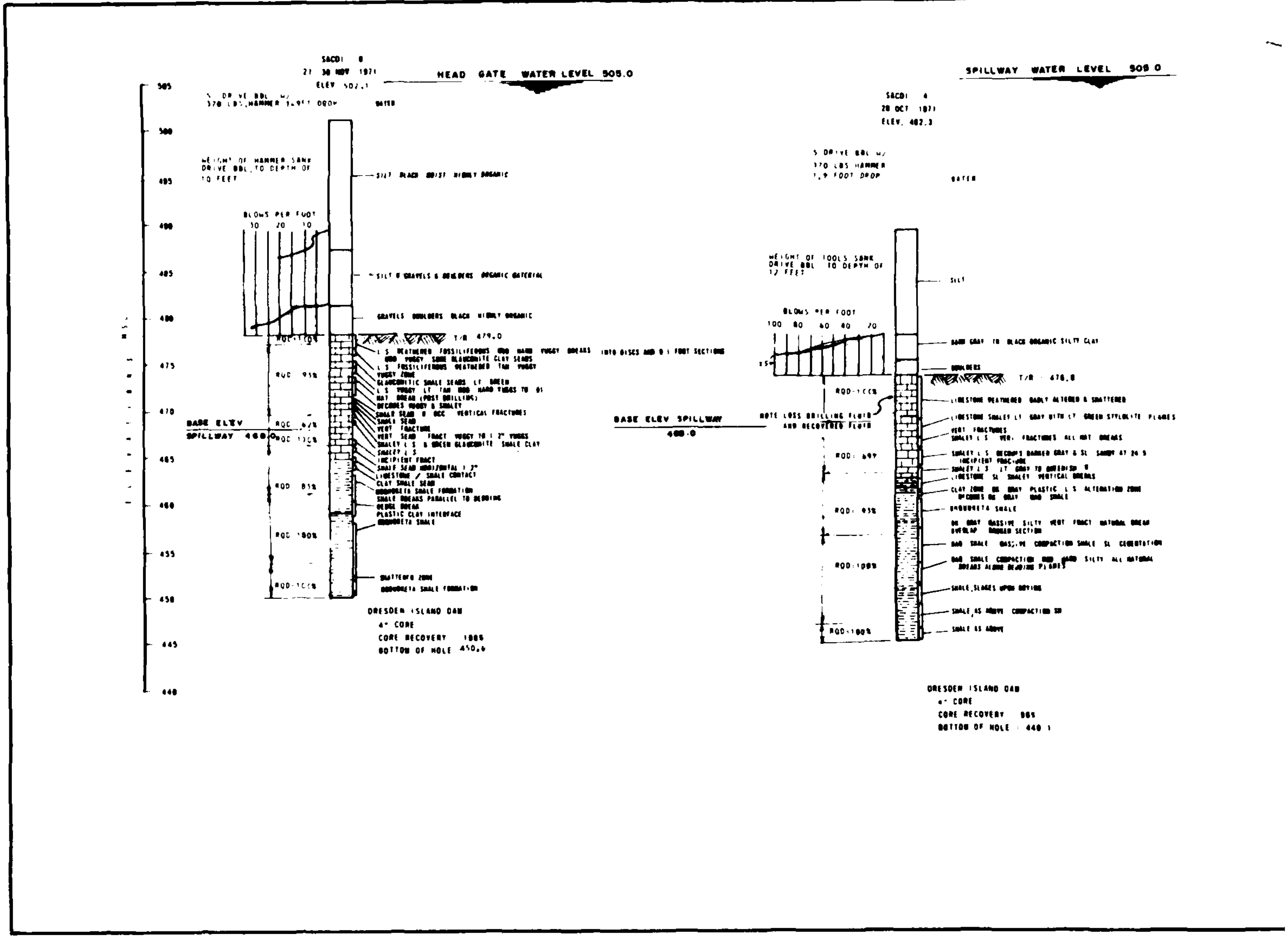
SEPT 72

SHEET 1 OF 2

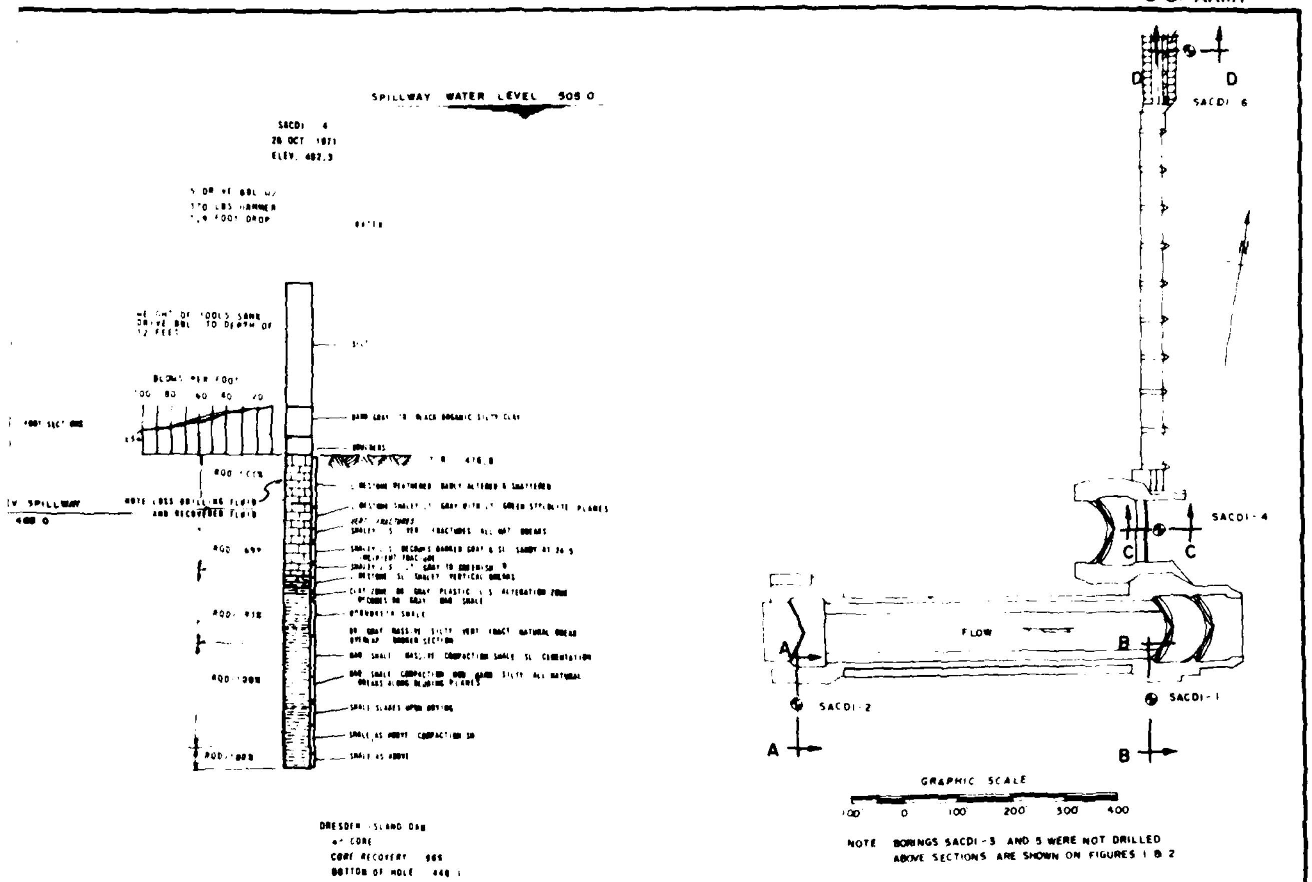
PLATE NO 3

PLATE 6

CORPS OF ENGINEERS



U.S. ARMY



NOTE: BORINGS SACDI-3 AND 5 WERE NOT DRILLED
ABOVE SECTIONS ARE SHOWN ON FIGURES 1 & 2

DRESDEN ISLAND LOCK & DAM
ILLINOIS WATERWAY, ILL.
LOGS OF BORINGS

SCALE AS SHOWN

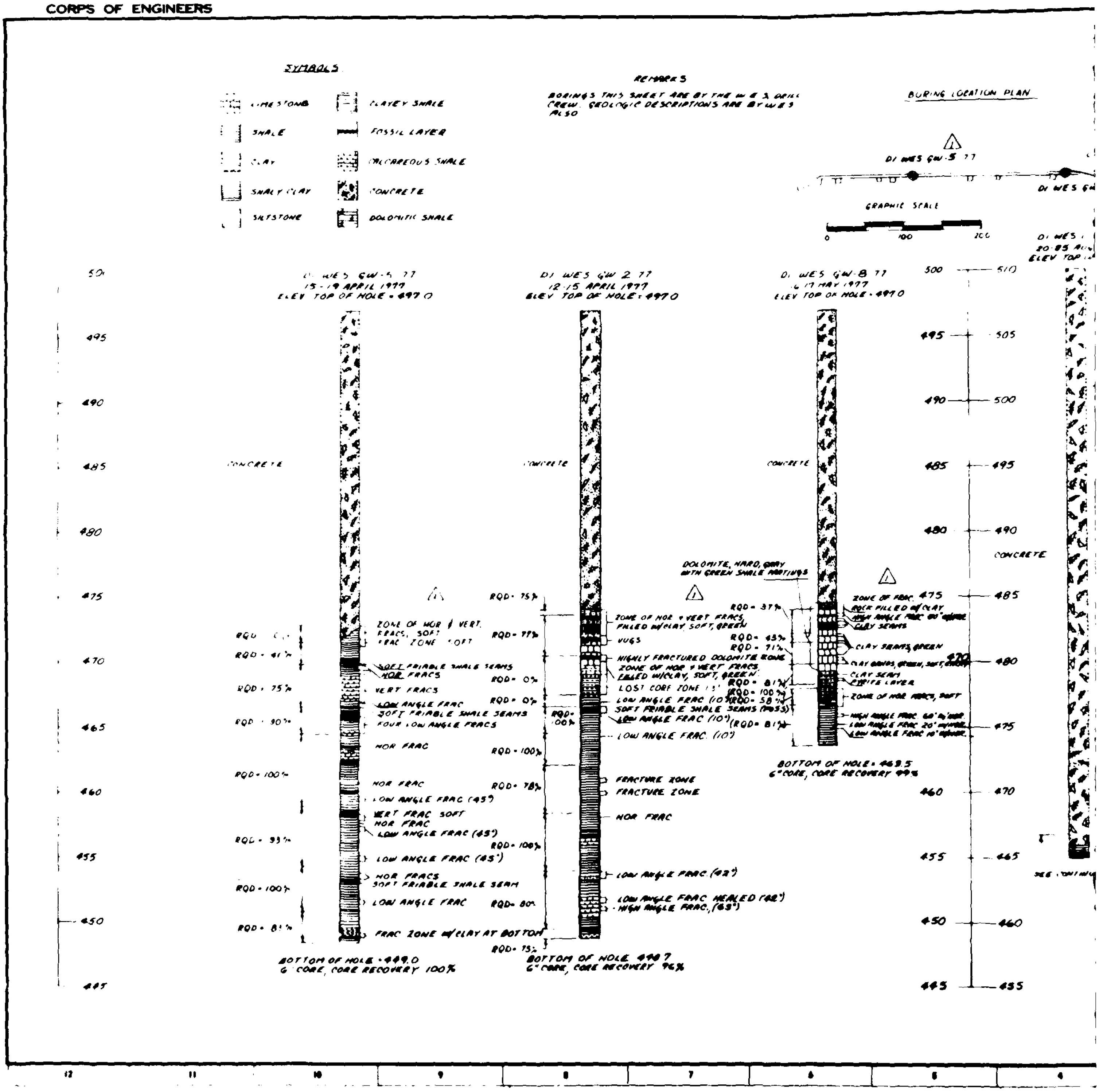
DEPARTMENT OF THE ARMY CHICAGO DISTRICT — CORPS OF ENGINEERS ENGINEERING DIVISION

31 JAN 72

SHEET 2 OF 2 — PLATE NO. 5

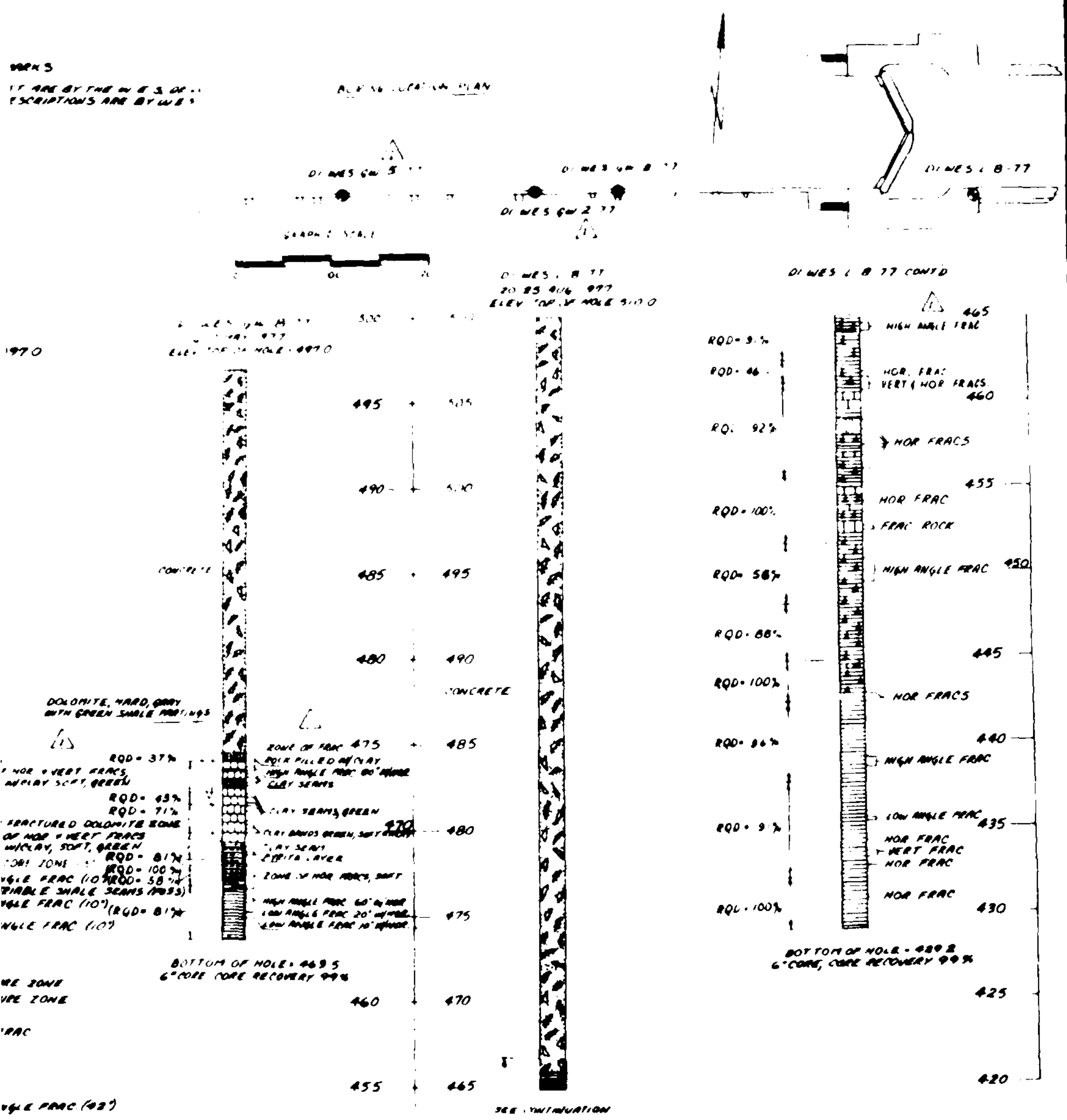
PLATE 7

CORPS OF ENGINEERS



12
11
10
9
8
7
6
5
4

U.S. ARMY



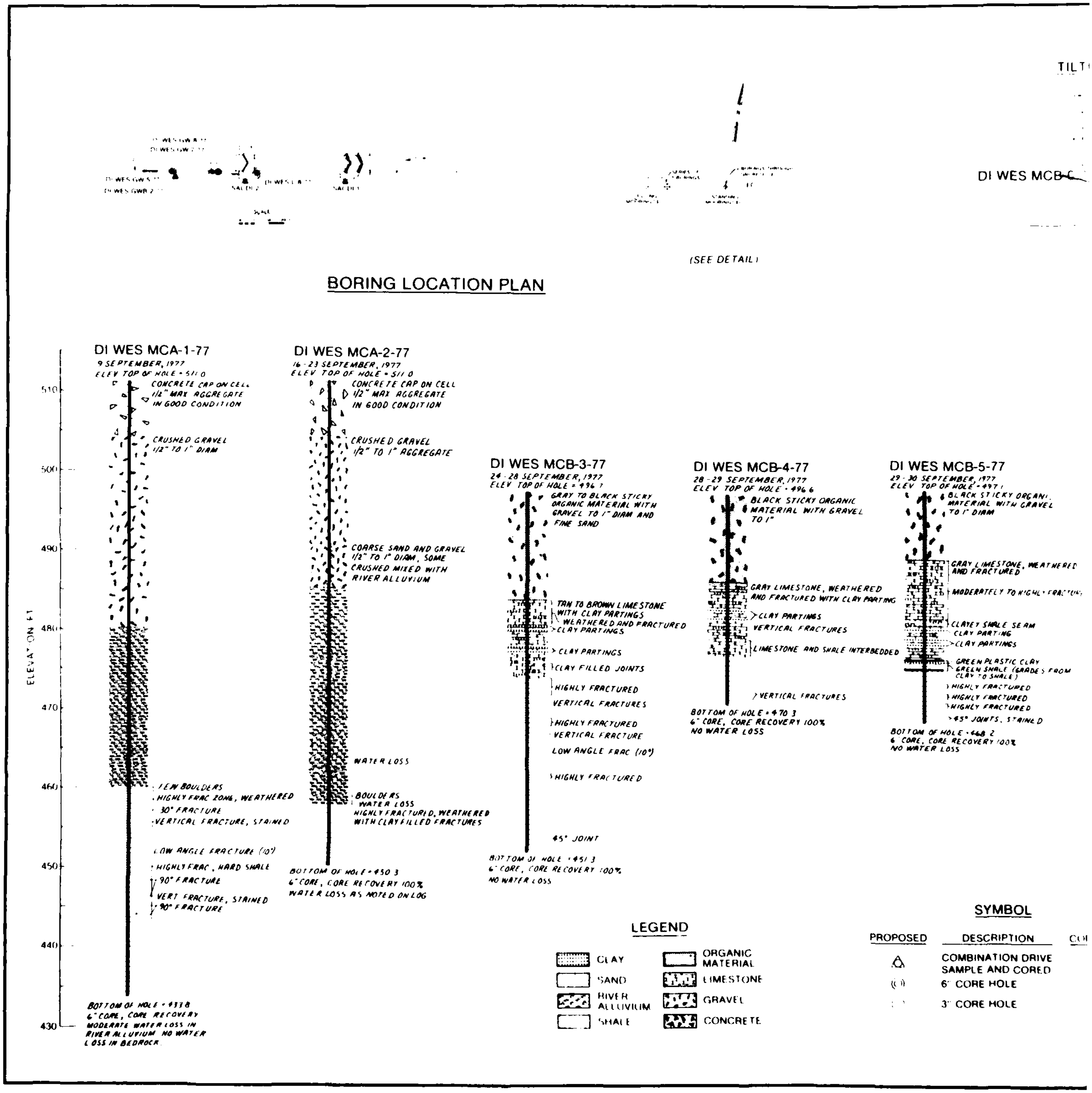
DRESDEN ISLAND
LOCK & DAM
ILLINOIS WATERWAY

LOG OF BORINGS

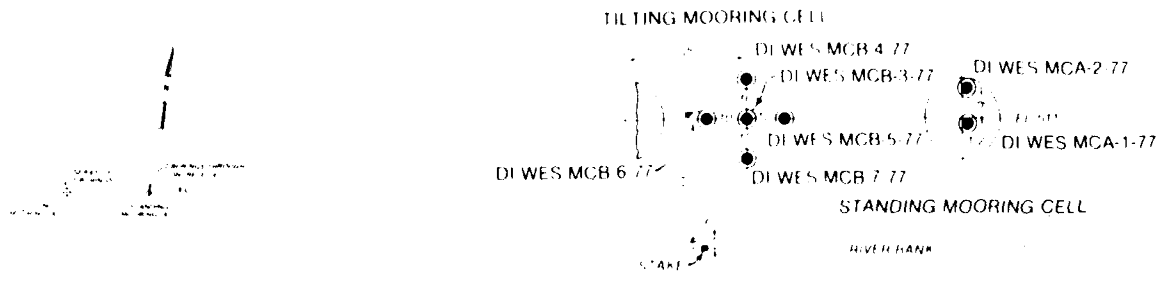
DATE MAY 1978

PLATE 8

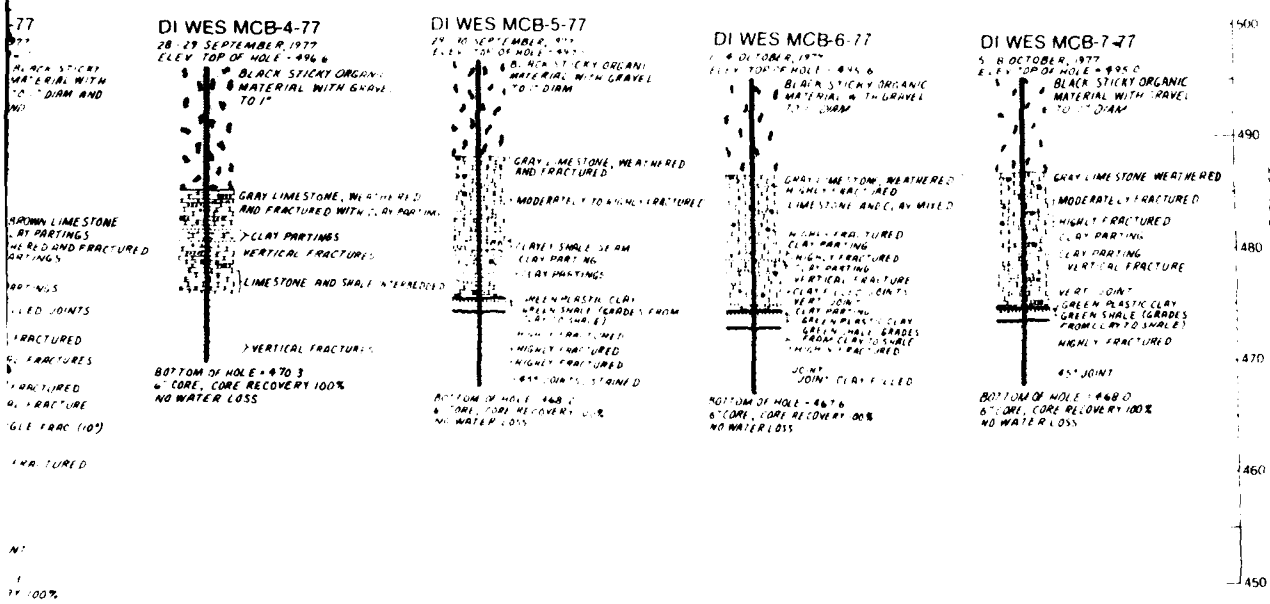
2



TILT
DI WES MCB-
(SEE DETAIL)
BORING LOCATION PLAN
DI WES MCA-1-77
9 SEPTEMBER, 1977
ELEV TOP OF HOLE = 511.0
CONCRETE CAP ON CELL 1/2" MAX AGGREGATE IN GOOD CONDITION
CRUSHED GRAVEL 1/2" TO 1" DIAM
FEW BOULDERS
HIGHLY FRAC ZONE, WEATHERED
30° FRACTURE
VERTICAL FRACTURE, STAINED
LOW ANGLE FRACTURE (10°)
HIGHLY FRAC, HARD SHALE
90° FRACTURE
VERT FRACTURE, STAINED
90° FRACTURE
BOTTOM OF HOLE = 433.8
6" CORE, CORE RECOVERY MODERATE WATER LOSS IN RIVER ALLUVIUM NO WATER LOSS IN BEDROCK
DI WES MCA-2-77
16-23 SEPTEMBER, 1977
ELEV TOP OF HOLE = 511.0
CONCRETE CAP ON CELL 1/2" MAX AGGREGATE IN GOOD CONDITION
CRUSHED GRAVEL 1/2" TO 1" AGGREGATE
COARSE SAND AND GRAVEL 1/2" TO 1" DIAM, SOME CRUSHED MIXED WITH RIVER ALLUVIUM
WATER LOSS
BOULDERS
WATER LOSS
HIGHLY FRACTURED, WEATHERED WITH CLAY FILLED FRACTURES
BOTTOM OF HOLE = 450.3
6" CORE, CORE RECOVERY 100%
WATER LOSS AS NOTED ON LOG
DI WES MCB-3-77
24-28 SEPTEMBER, 1977
ELEV TOP OF HOLE = 496.7
GRAY TO BLACK STICKY ORGANIC MATERIAL WITH GRAVEL TO 1" DIAM AND FINE SAND
TAN TO BROWN LIMESTONE WITH CLAY PARTINGS
WEATHERED AND FRACTURED
CLAY PARTINGS
CLAY PARTINGS
CLAY FILLED JOINTS
HIGHLY FRACTURED
VERTICAL FRACTURES
HIGHLY FRACTURED
VERTICAL FRACTURE
LOW ANGLE FRAC (10°)
HIGHLY FRACTURED
45° JOINT
BOTTOM OF HOLE = 451.3
6" CORE, CORE RECOVERY 100%
NO WATER LOSS
DI WES MCB-4-77
28-29 SEPTEMBER, 1977
ELEV TOP OF HOLE = 496.6
BLACK STICKY ORGANIC MATERIAL WITH GRAVEL TO 1"
GRAY LIMESTONE, WEATHERED AND FRACTURED WITH CLAY PARTING
CLAY PARTINGS
VERTICAL FRACTURES
LIMESTONE AND SHALE INTERBEDDED
VERTICAL FRACTURES
BOTTOM OF HOLE = 470.3
6" CORE, CORE RECOVERY 100%
NO WATER LOSS
DI WES MCB-5-77
29-30 SEPTEMBER, 1977
ELEV TOP OF HOLE = 497.1
BLACK STICKY ORGANIC MATERIAL WITH GRAVEL TO 1" DIAM
GRAY LIMESTONE, WEATHERED AND FRACTURED
MODERATELY TO HIGHLY FRACTURED
CLAYEY SHALE SEAM
CLAY PARTING
CLAY PARTINGS
GREEN PLASTIC CLAY
GREEN SHALE (GRADES FROM CLAY TO SHALE)
HIGHLY FRACTURED
HIGHLY FRACTURED
HIGHLY FRACTURED
45° JOINTS, STAINED
BOTTOM OF HOLE = 468.2
6" CORE, CORE RECOVERY 100%
NO WATER LOSS
ELEVATION, FT
510
500
490
480
470
460
450
440
430
LEGEND
CLAY
SAND
RIVER ALLUVIUM
SHALE
ORGANIC MATERIAL
LIMESTONE
GRAVEL
CONCRETE
SYMBOL
PROPOSED
DESCRIPTION
COMBINATION DRIVE SAMPLE AND CORED
6" CORE HOLE
3" CORE HOLE



DETAIL



LEGEND

CLAY — ORGANIC MATERIAL
SAND — LIMESTONE
RIVER ALLUVIUM — GRAVEL
SHALE — CONCRETE

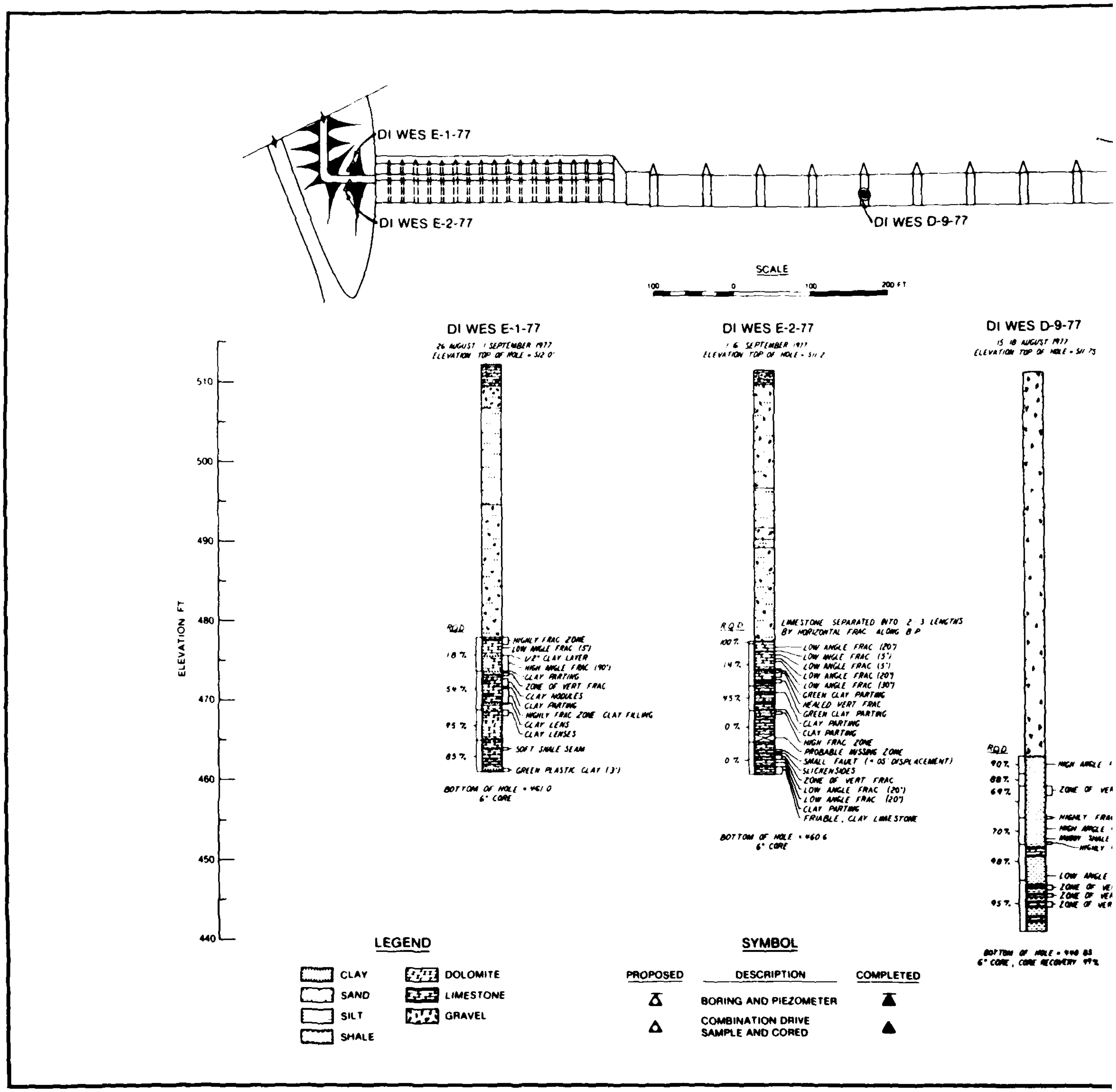
SYMBOL

PROPOSED	DESCRIPTION	COMPLETED
	COMBINATION DRIVE SAMPLE AND CORED	
	6" CORE HOLE	
	3" CORE HOLE	

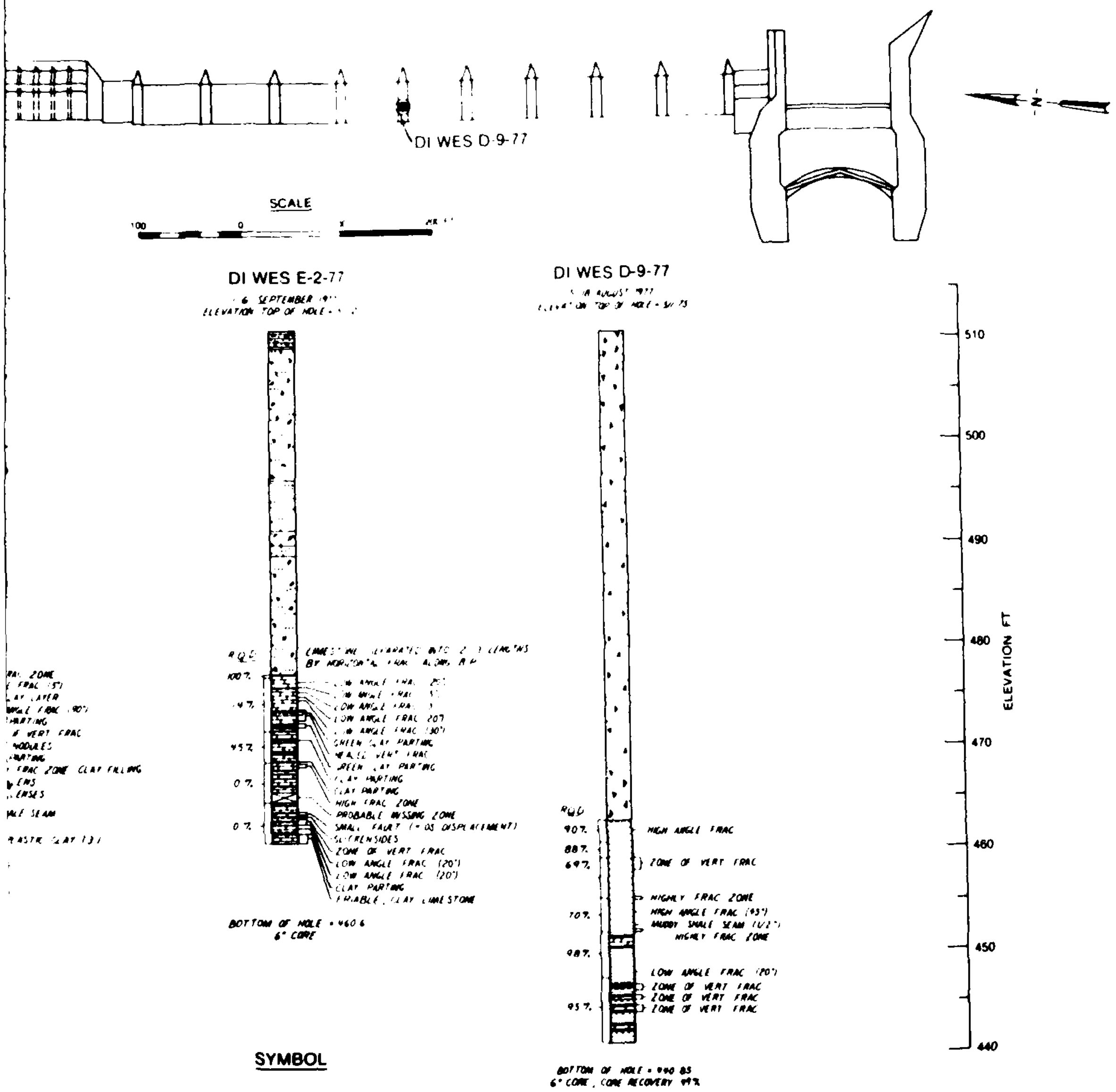
MOORING CELLS
DRESDEN ISLAND LOCK AND DAM
ILLINOIS WATERWAY

LOG OF BORINGS
UPSTREAM MOST MOORING CELLS

PLATE 9



DI WES E-1-77
DI WES E-2-77
DI WES D-9-77
SCALE
100 0 100 200 FT
ELEVATION FT
510
500
490
480
470
460
450
440
DI WES E-1-77
26 AUGUST 1 SEPTEMBER 1977
ELEVATION TOP OF HOLE = 512 0'
RQD
18 %
54 %
95 %
85 %
HIGHLY FRAC ZONE
LOW ANGLE FRAC (5°)
1/2" CLAY LAYER
HIGH ANGLE FRAC (90°)
CLAY PARTING
ZONE OF VERT FRAC
CLAY NODULES
CLAY PARTING
HIGHLY FRAC ZONE CLAY FILLING
CLAY LENS
CLAY LENSES
SOFT SHALE SEAM
GREEN PLASTIC CLAY (3")
BOTTOM OF HOLE = 461 0
6" CORE
DI WES E-2-77
1 6 SEPTEMBER 1977
ELEVATION TOP OF HOLE = 511 2
RQD
100 %
14 %
45 %
0 %
0 %
LIMESTONE SEPARATED INTO 2 3 LENGTHS
BY HORIZONTAL FRAC ALONG B P
LOW ANGLE FRAC (20°)
LOW ANGLE FRAC (5°)
LOW ANGLE FRAC (5°)
LOW ANGLE FRAC (20°)
LOW ANGLE FRAC (30°)
GREEN CLAY PARTING
HEALED VERT FRAC
GREEN CLAY PARTING
CLAY PARTING
CLAY PARTING
HIGH FRAC ZONE
PROBABLE MISSING ZONE
SMALL FAULT (+ 05' DISPLACEMENT)
SLICKENSIDES
ZONE OF VERT FRAC
LOW ANGLE FRAC (20°)
LOW ANGLE FRAC (20°)
CLAY PARTING
FRIABLE, CLAY LIMESTONE
BOTTOM OF HOLE = 460 6
6" CORE
DI WES D-9-77
15 18 AUGUST 1977
ELEVATION TOP OF HOLE = 511 75
RQD
90 %
88 %
69 %
70 %
98 %
95 %
BOTTOM OF HOLE = 440 85
6" CORE, CORE RECOVERY 99%
LEGEND
CLAY
SAND
SILT
SHALE
DOLOMITE
LIMESTONE
GRAVEL
SYMBOL
PROPOSED
DESCRIPTION
COMPLETED
BORING AND PIEZOMETER
COMBINATION DRIVE
SAMPLE AND CORED



SYMBOL

PROPOSED	DESCRIPTION	COMPLETED
Δ	BORING AND PIEZOMETER	▲
Δ	COMBINATION DRIVE SAMPLE AND CORED	▲

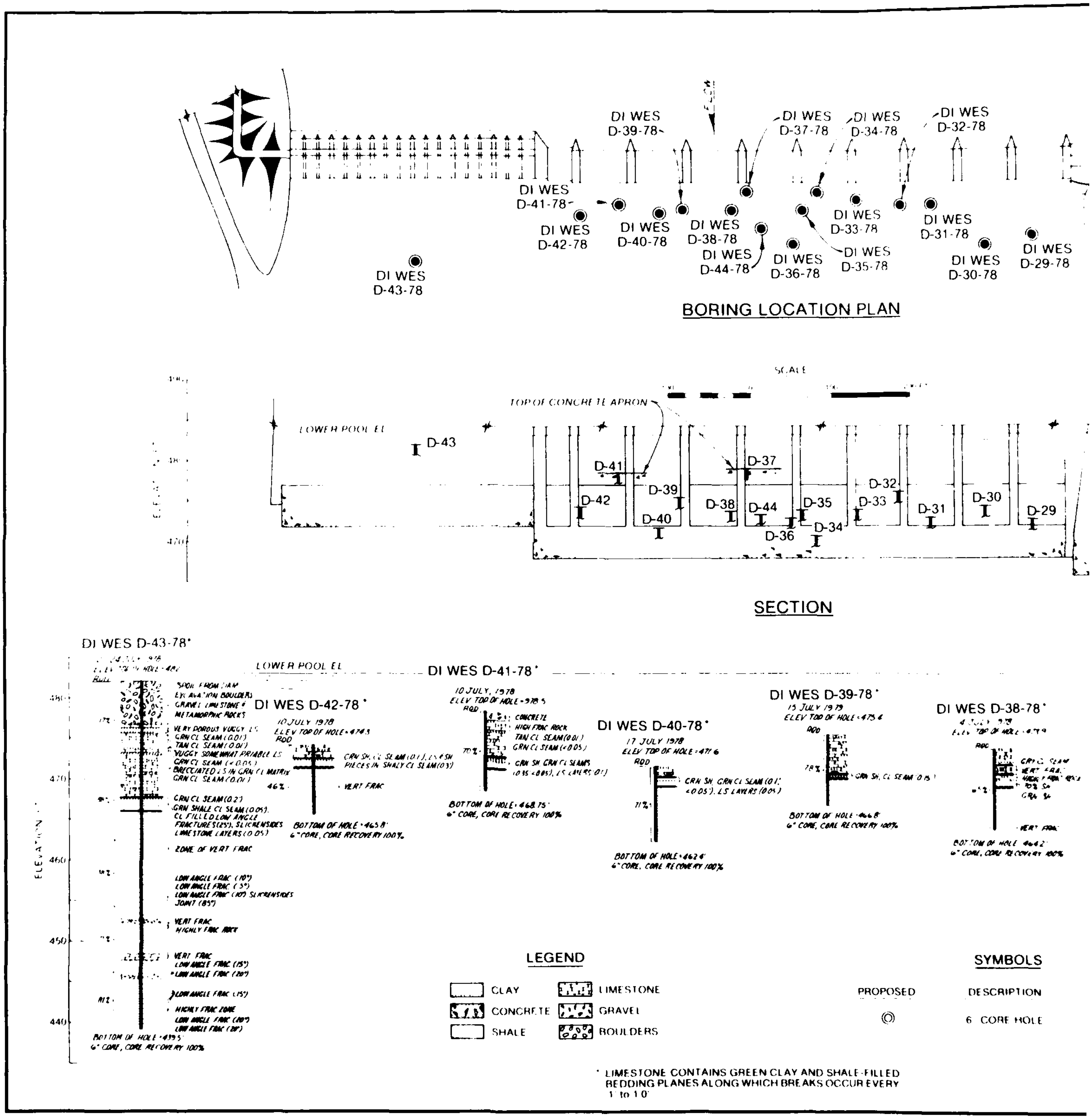
COMPLIANCE PHASE

DRESDEN ISLAND LOCK AND DAM
ILLINOIS WATERWAY

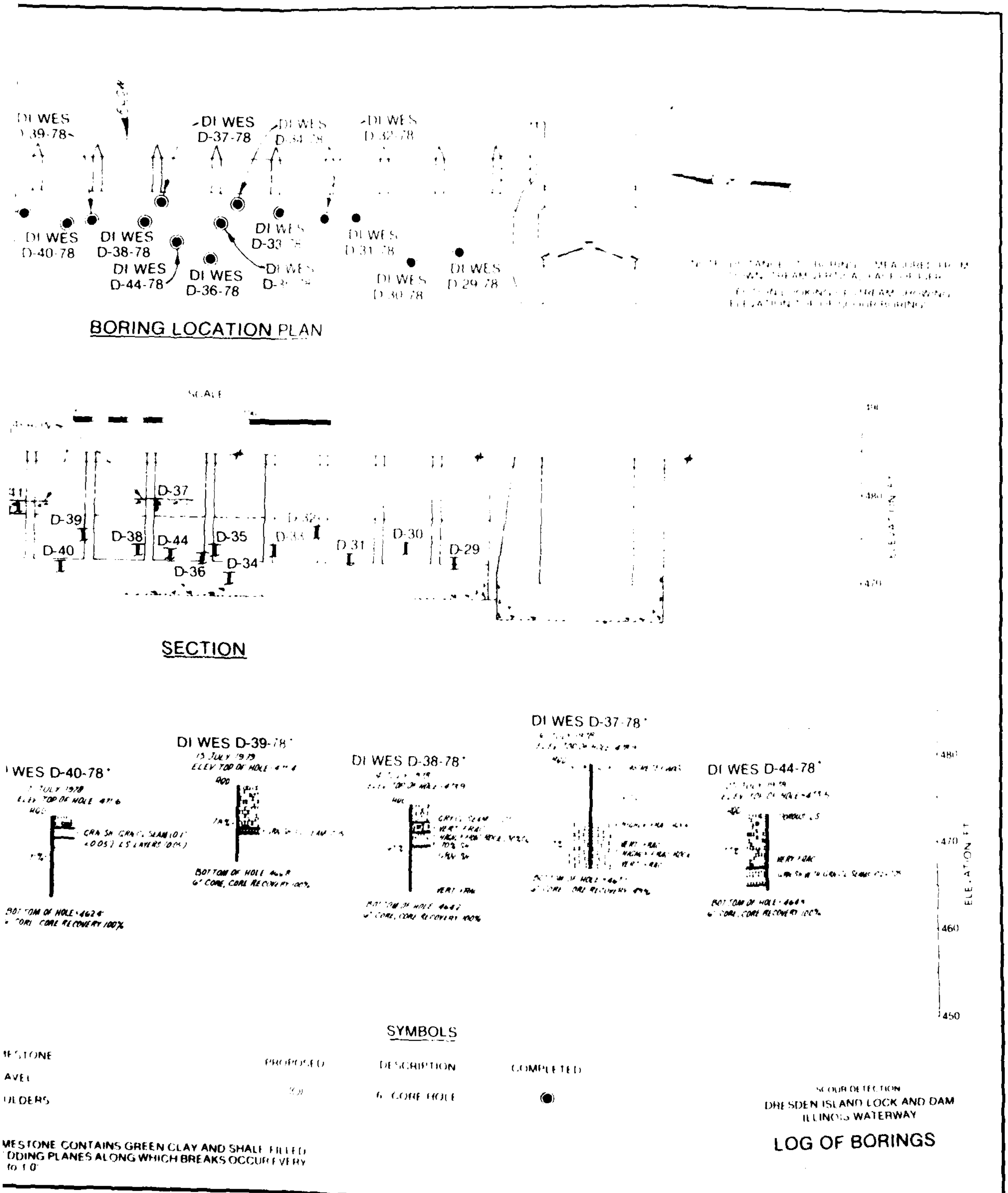
LOG OF BORINGS

AUGUST, 1978

PLATE 10

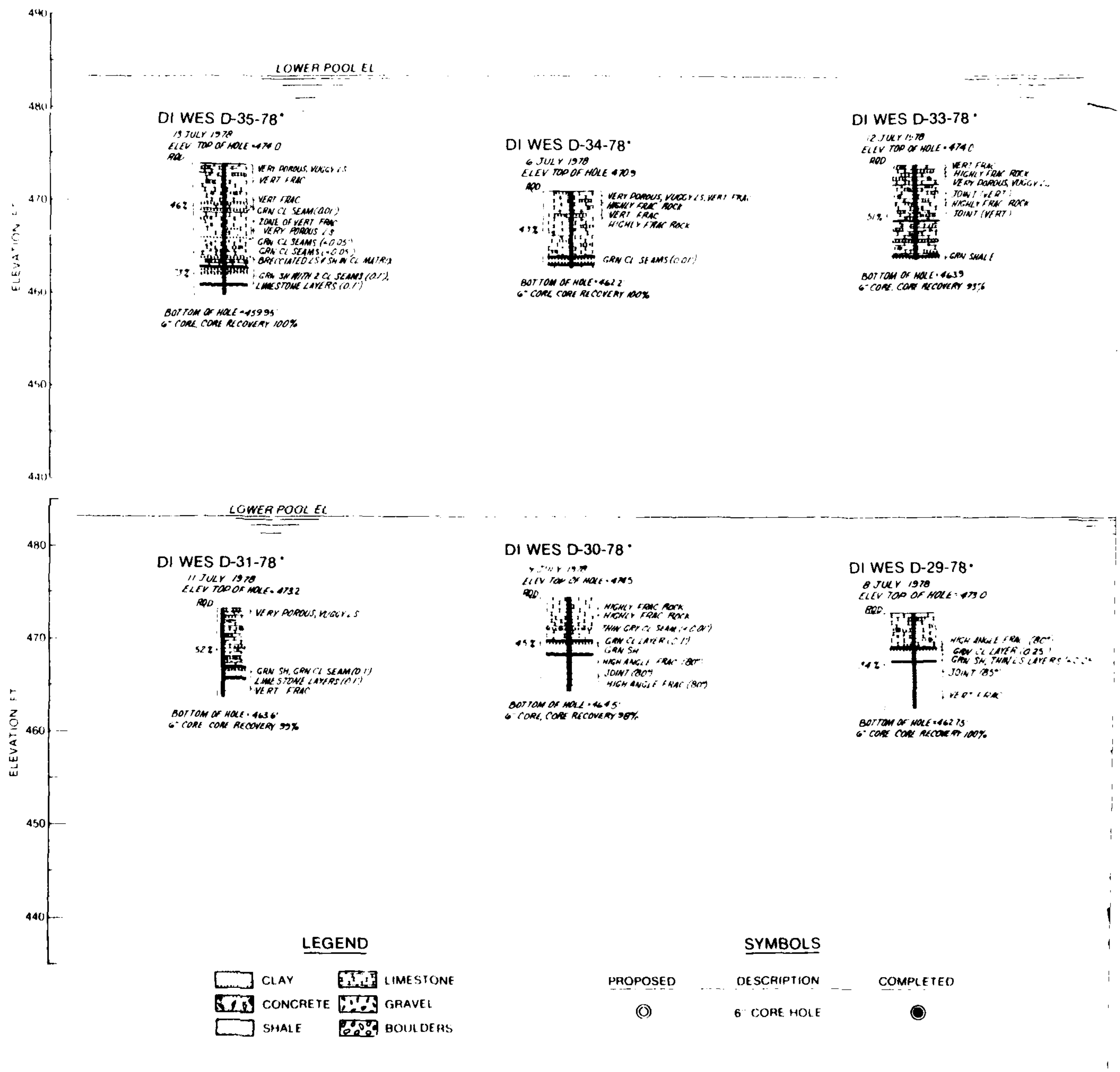


BORING LOCATION PLAN
DI WES D-29-78
DI WES D-30-78
DI WES D-31-78
DI WES D-32-78
DI WES D-33-78
DI WES D-34-78
DI WES D-35-78
DI WES D-36-78
DI WES D-37-78
DI WES D-38-78
DI WES D-39-78
DI WES D-40-78
DI WES D-41-78
DI WES D-42-78
DI WES D-43-78
DI WES D-44-78
SCALE
TOP OF CONCRETE APRON
LOWER POOL EL
SECTION
ELEVATION
DI WES D-43-78*
DI WES D-42-78*
DI WES D-41-78*
DI WES D-40-78*
DI WES D-39-78*
DI WES D-38-78*
BOTTOM OF HOLE 468.75
6" CORE, CORE RECOVERY 100%
LEGEND
CLAY
CONCRETE
SHALE
LIMESTONE
GRAVEL
BOULDERS
SYMBOLS
PROPOSED
DESCRIPTION
6" CORE HOLE
* LIMESTONE CONTAINS GREEN CLAY AND SHALE-FILLED BEDDING PLANES ALONG WHICH BREAKS OCCUR EVERY 1 to 1.0'

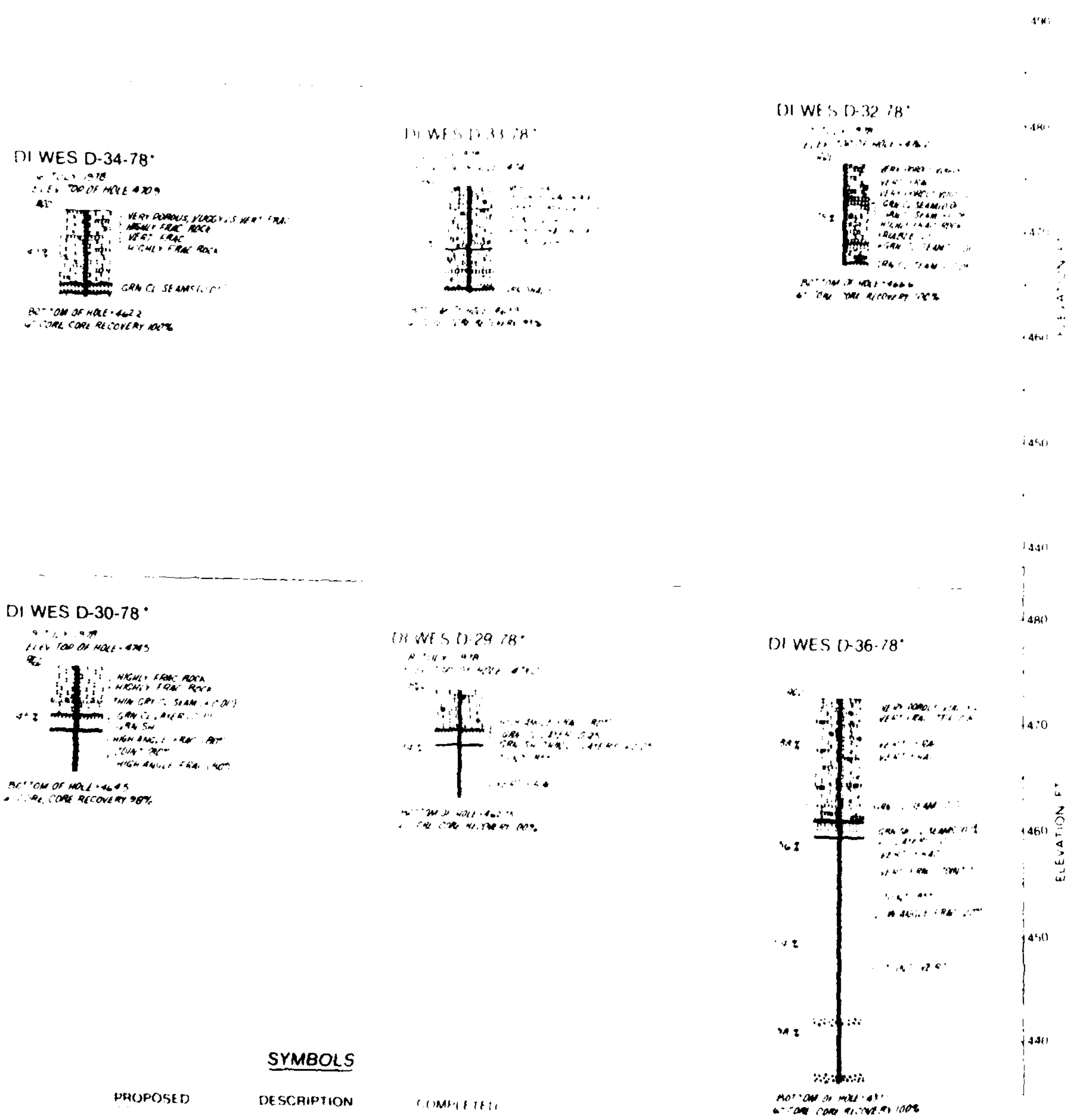


DI WES D-37-78
DI WES D-40-78
DI WES D-38-78
DI WES D-44-78
DI WES D-36-78
DI WES D-29-78
BORING LOCATION PLAN
SCALE
D-37
D-39
D-38
D-40
D-44
D-35
D-36
D-34
D-31
D-30
D-29
SECTION
DI WES D-39-78
DI WES D-38-78
DI WES D-37-78
DI WES D-44-78
480
470
460
450
ELEVATION FT
SYMBOLS
PROPOSED
DESCRIPTION
COMPLETED
6" CORE HOLE
SCOUR DETECTION
DRESDEN ISLAND LOCK AND DAM
ILLINOIS WATERWAY
LOG OF BORINGS

PLATE 11



* LIMESTONE CONTAINS GREEN CLAY AND SHALE FILLED BEDDING PLANES ALONG WHICH BREAKS OCCUR EVERY 1' to 1.0'



SYMBOLS

PROPOSED	DESCRIPTION	COMPLETED
◎	6" CORE HOLE	●

...TAINS GREEN CLAY AND SHALE FILLED
...S ALONG WHICH BREAKS OCCUR EVERY

SCOUR DETECTION
DRESDEN ISLAND LOCK AND DAM
ILLINOIS WATERWAY

LOG OF BORINGS

PLATE 12b

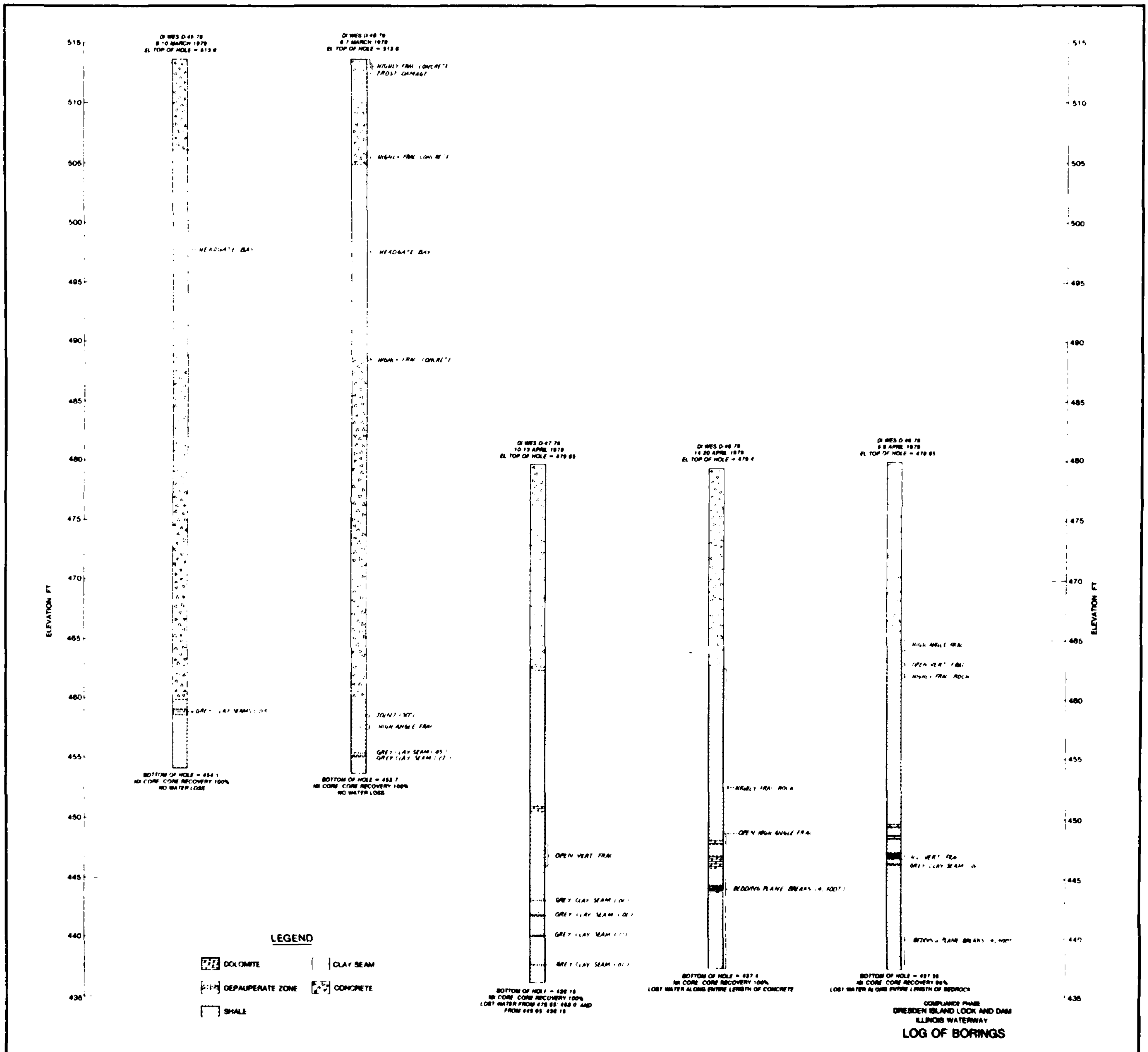
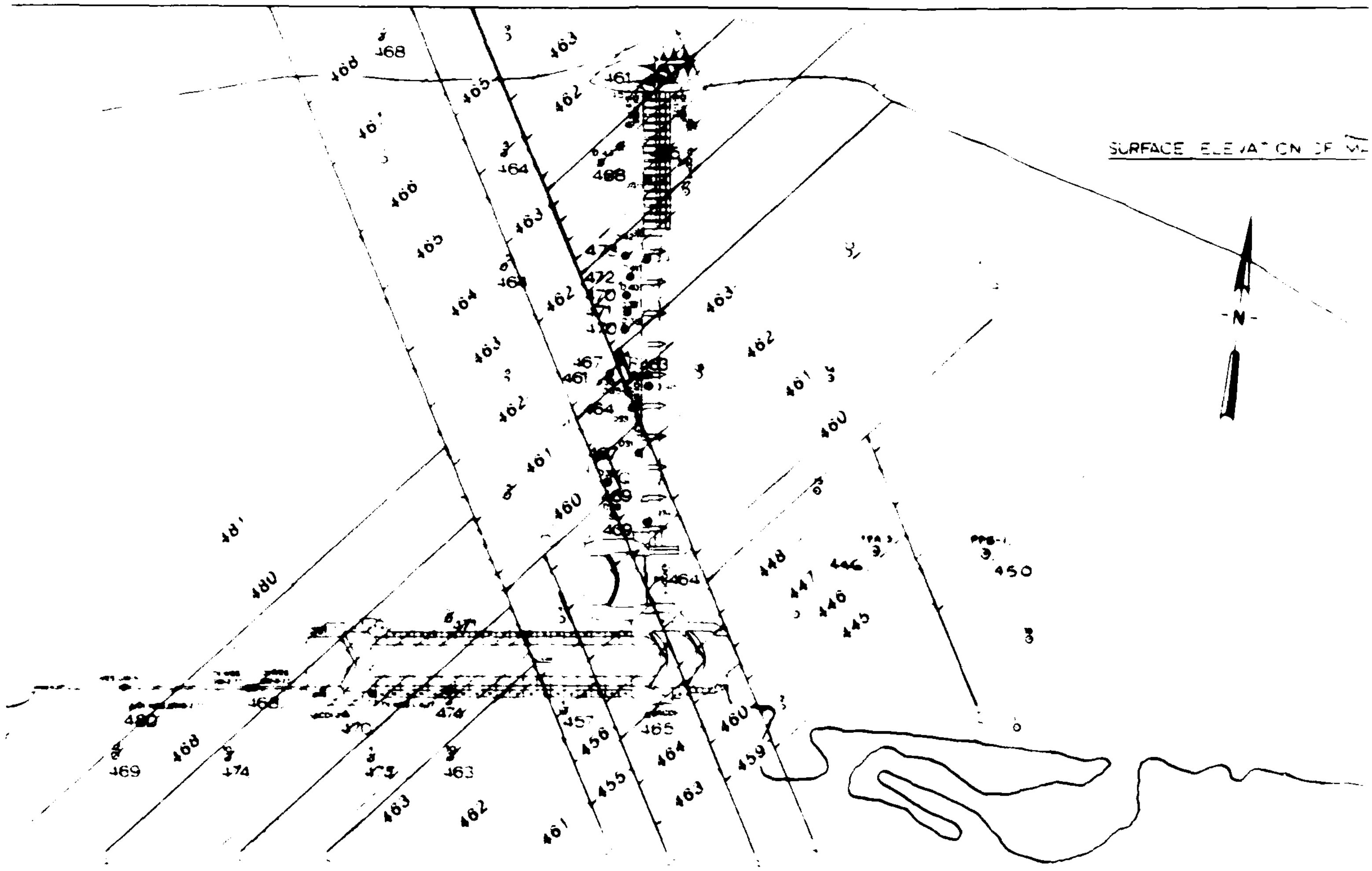

ELEVATION FT
BOTTOM OF HOLE = 454.1
NX CORE. CORE RECOVERY 100%
NO WATER LOSS
HEADGATE BAY
HIGHLY FRAC CONCRETE
FROST DAMAGE
HIGHLY FRAC CONCRETE
HIGH ANGLE FRAC
BOTTOM OF HOLE = 453.7
NX CORE. CORE RECOVERY 100%
NO WATER LOSS
OPEN VERT FRAC
GREY CLAY SEAM
OPEN HIGH ANGLE FRAC
BEDDING PLANE BREAKS
HIGHLY FRAC ROCK
HIGH ANGLE FRAC
OPEN VERT FRAC
LOST WATER ALONG ENTIRE LENGTH OF CONCRETE
LOST WATER ALONG ENTIRE LENGTH OF BEDROCK
LEGEND
DOLOMITE
CLAY SEAM
DEPAUPERATE ZONE
CONCRETE
SHALE
COMPLIANCE PHASE
DRESDEN ISLAND LOCK AND DAM
ILLINOIS WATERWAY
LOG OF BORINGS

PLATE 12b



SYMBOL

Construction borings

IDC & WES borings

Fault

Borings contain breccia/clay w/ Sh & LS pieces & slickensides

Normal fault traces from Illinois Geological Survey

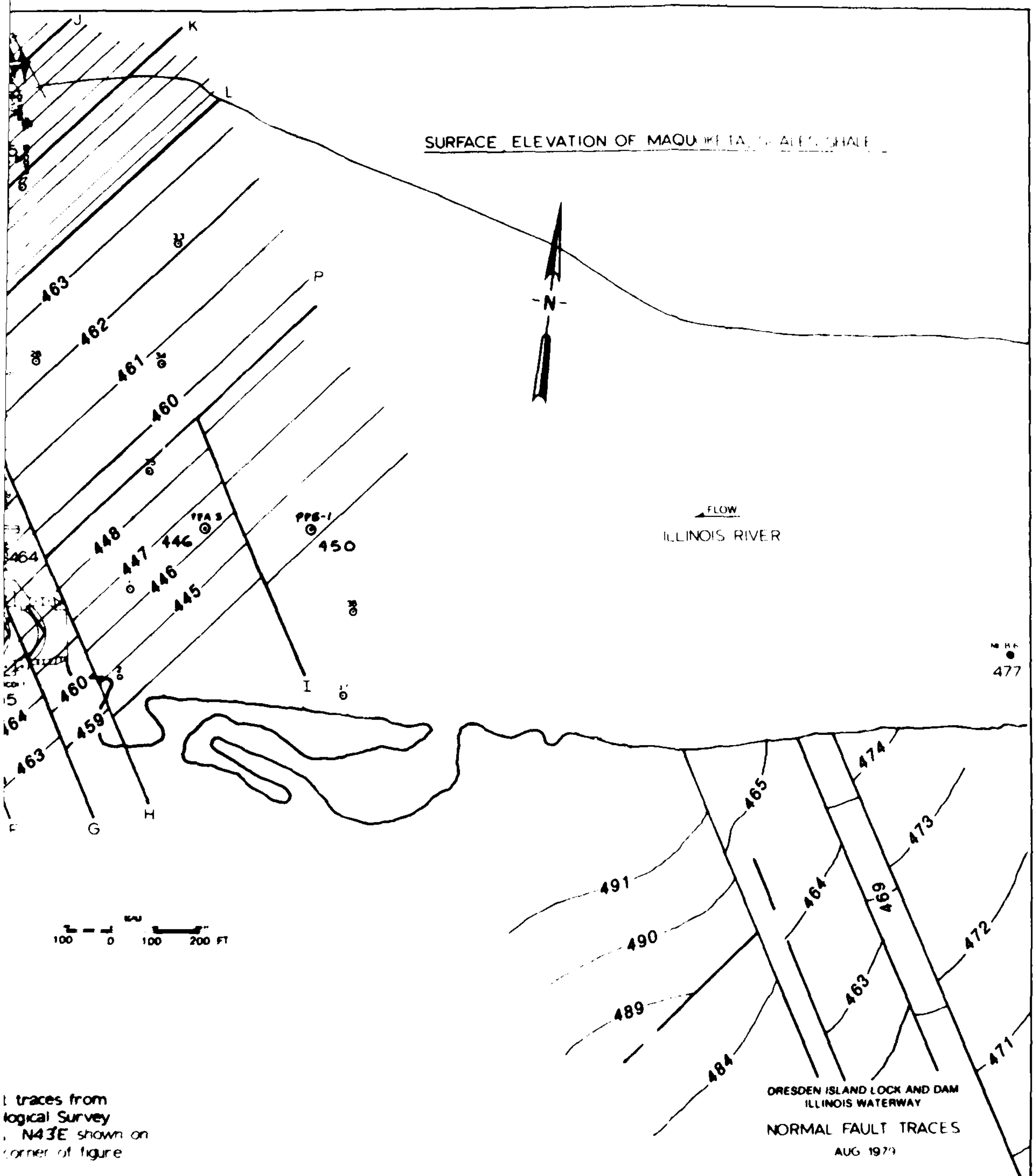
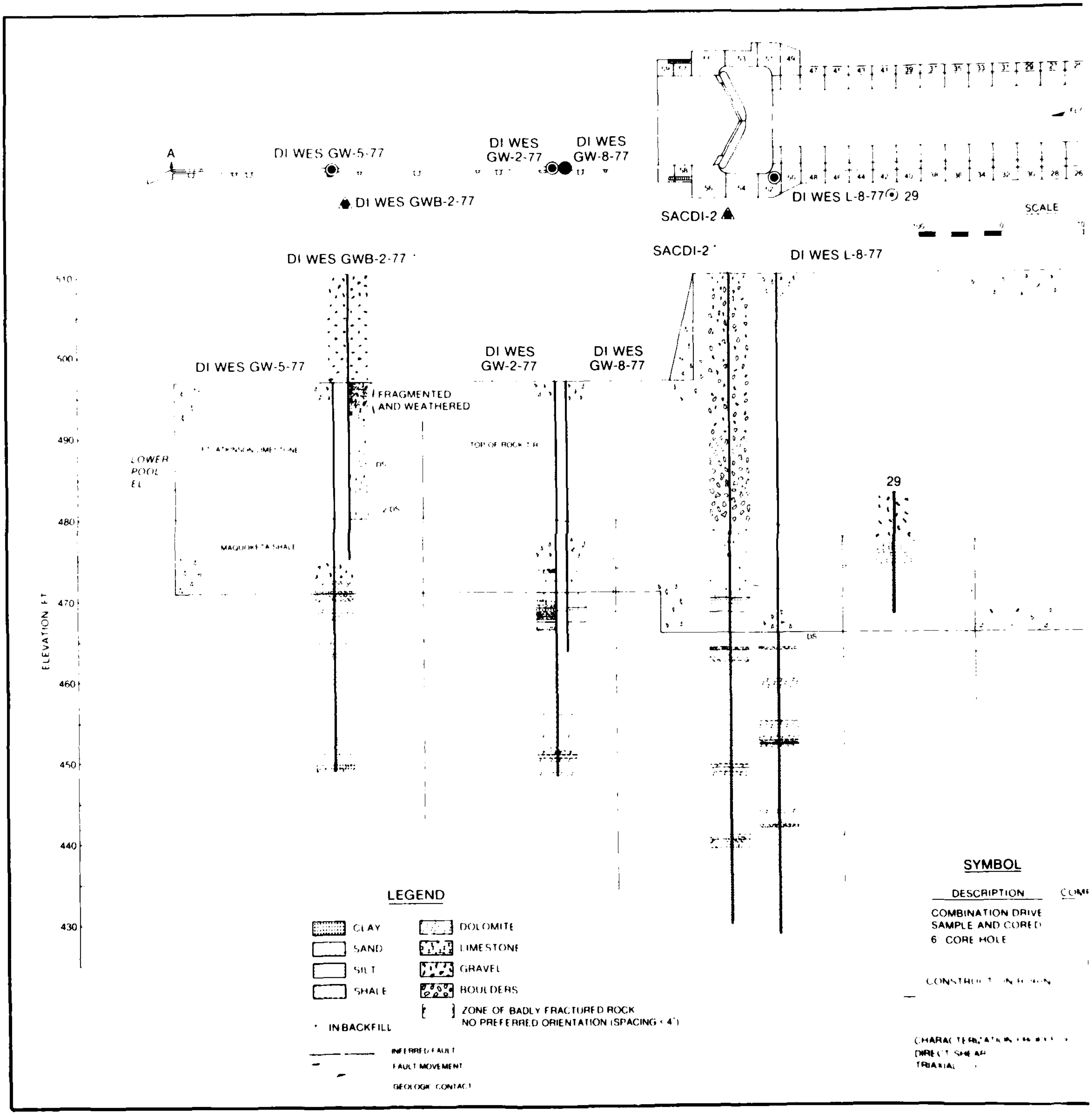
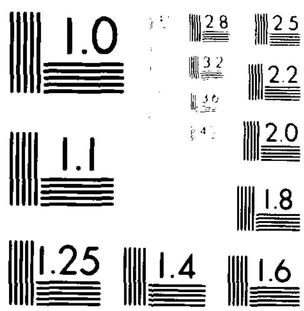

SURFACE ELEVATION OF MAQU
-N-
FLOW
ILLINOIS RIVER
450
477
463
462
461
460
448
447
446
446
445
460
459
463
464
474
473
472
471
469
465
464
463
491
490
489
484
J
K
L
P
I
G
H
100 0 100 200 FT
traces from
logical Survey
N43E shown on
corner of figure
DRESDEN ISLAND LOCK AND DAM
ILLINOIS WATERWAY
NORMAL FAULT TRACES
AUG 1979

PLATE 12c

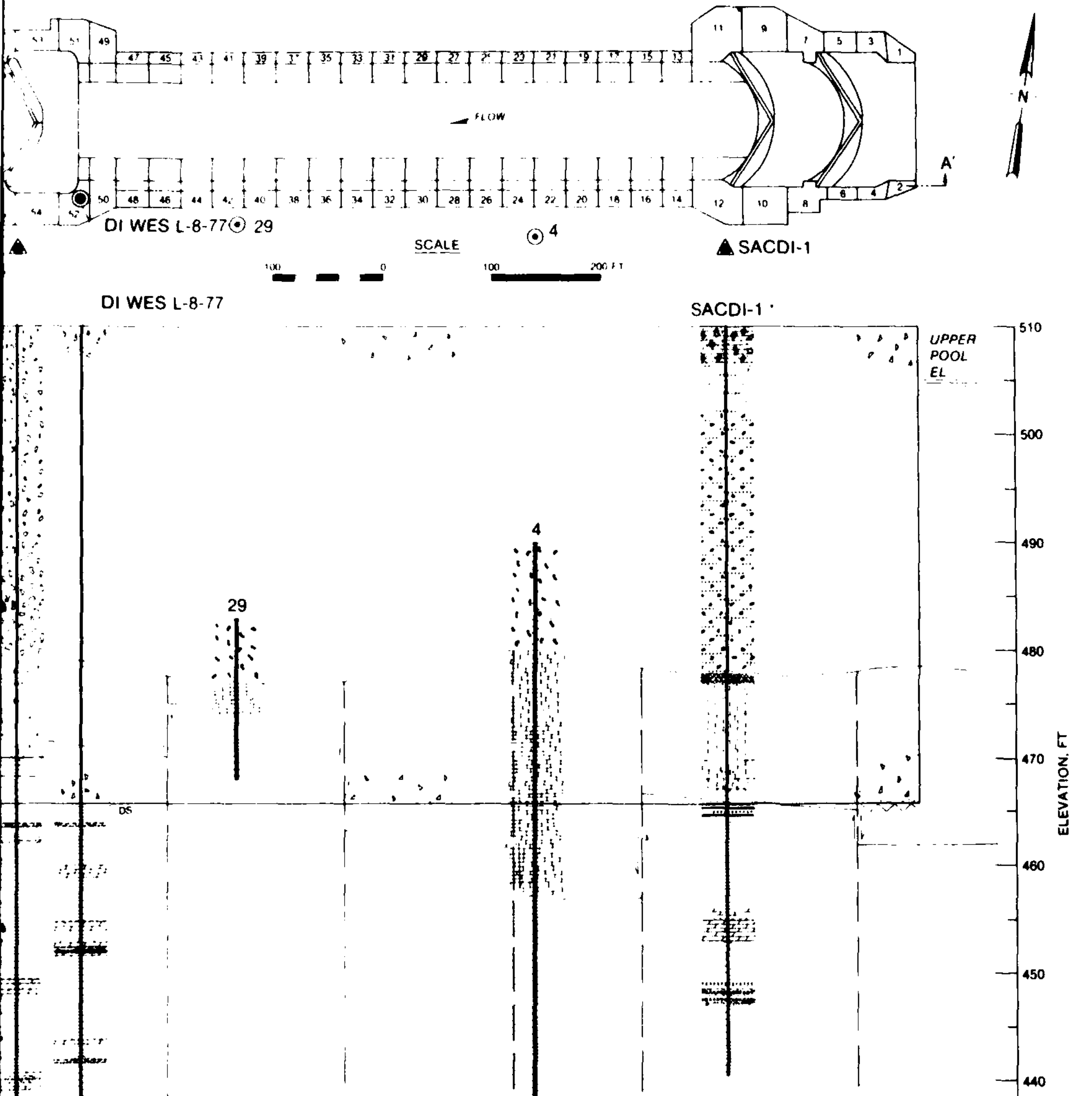

A
DI WES GW-5-77
DI WES GW-2-77
DI WES GW-8-77
DI WES GWB-2-77
SACDI-2
DI WES L-8-77
29
SCALE
DI WES GWB-2-77
SACDI-2
DI WES L-8-77
DI WES GW-5-77
DI WES GW-2-77
DI WES GW-8-77
FRAGMENTED AND WEATHERED
TOP OF ROCK
LOWER POOL EL
MAQUOKETA SHALE
DS
ELEVATION, FT
510
500
490
480
470
460
450
440
430
29
LEGEND
CLAY
SAND
SILT
SHALE
DOLOMITE
LIMESTONE
GRAVEL
BOULDERS
ZONE OF BADLY FRACTURED ROCK
NO PREFERRED ORIENTATION (SPACING < 4')
IN BACKFILL
INFERRED FAULT
FAULT MOVEMENT
GEOLOGIC CONTACT
SYMBOL
DESCRIPTION
COMBINATION DRIVE SAMPLE AND CORED 6" CORE HOLE
DIRECT SHEAR
TRIAXIAL

AD-A098 613 ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG--ETC F/G 13/13
CONCRETE AND ROCK TESTS, MAJOR REHABILITATION OF DRESDEN ISLAND--ETC(U)
MAR 81 R L STOWE, B A PAVLOV
UNCLASSIFIED WES/MP/SL-81-1 NL

2 OF 3
AD A098613



MICROCOPY RESOLUTION TEST CHART



SYMBOL

DESCRIPTION	COMPLETED
COMBINATION DRIVE SAMPLE AND CORED	▲
6" CORE HOLE	◉
	●
CONSTRUCTION BORING	⊙

CHARACTERIZATION PROPERTIES CP UC , V, , γ
DIRECT SHEAR DS
TRIAXIAL TX

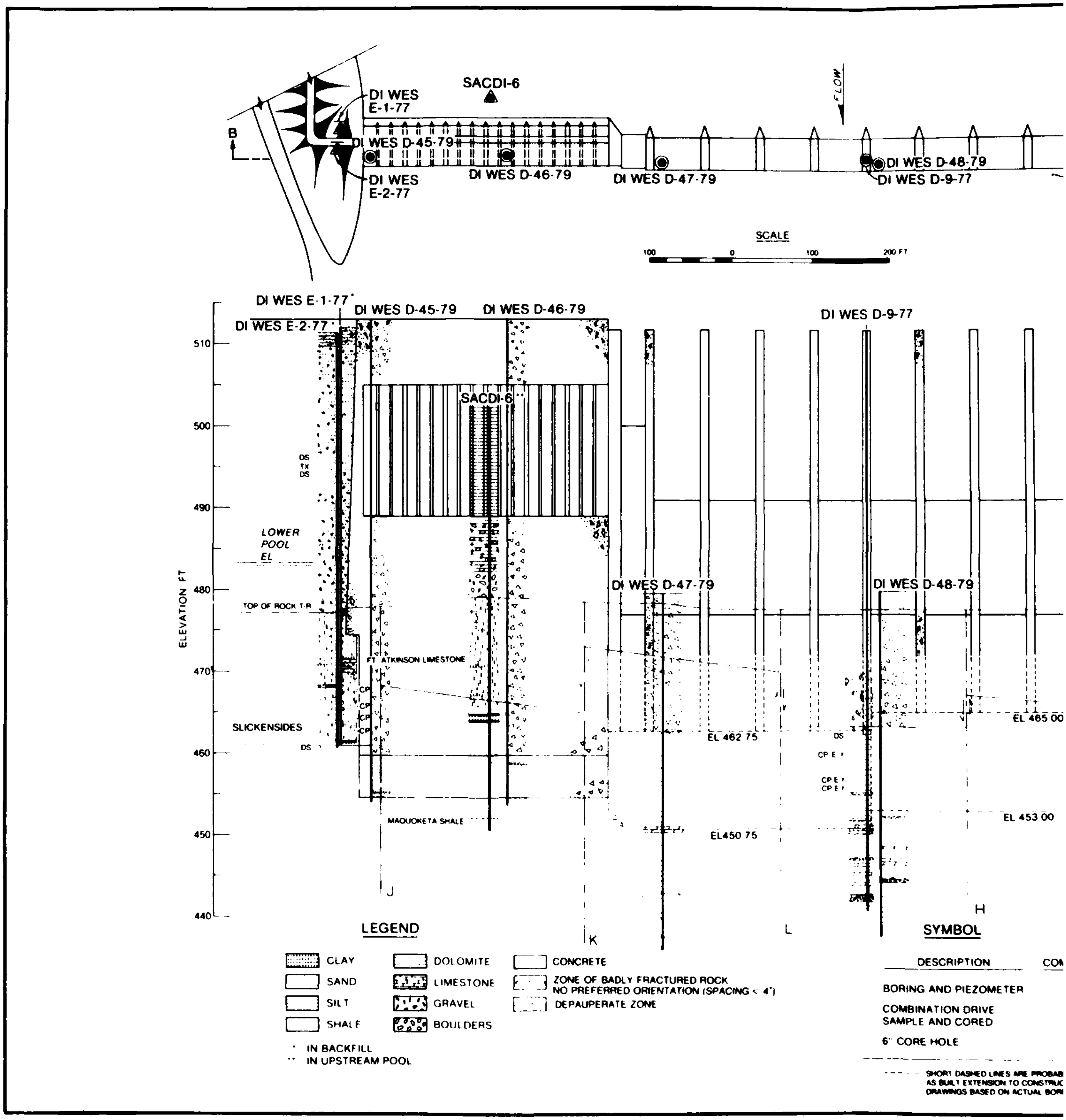
REHABILITATION PHASE
DRESDEN ISLAND LOCK AND DAM
ILLINOIS WATERWAY

GEOLOGIC CROSS SECTION

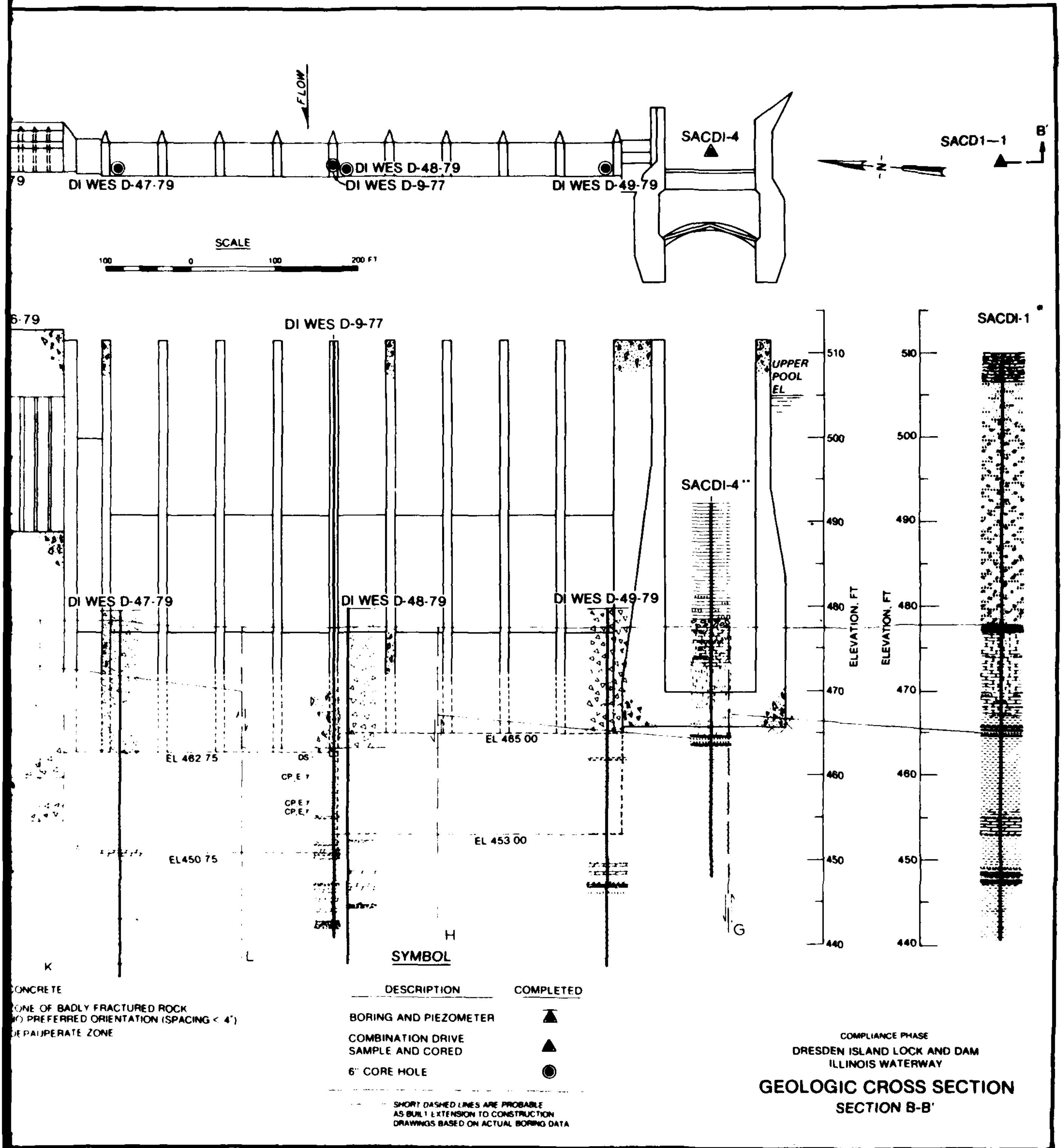
SECTION A-A'

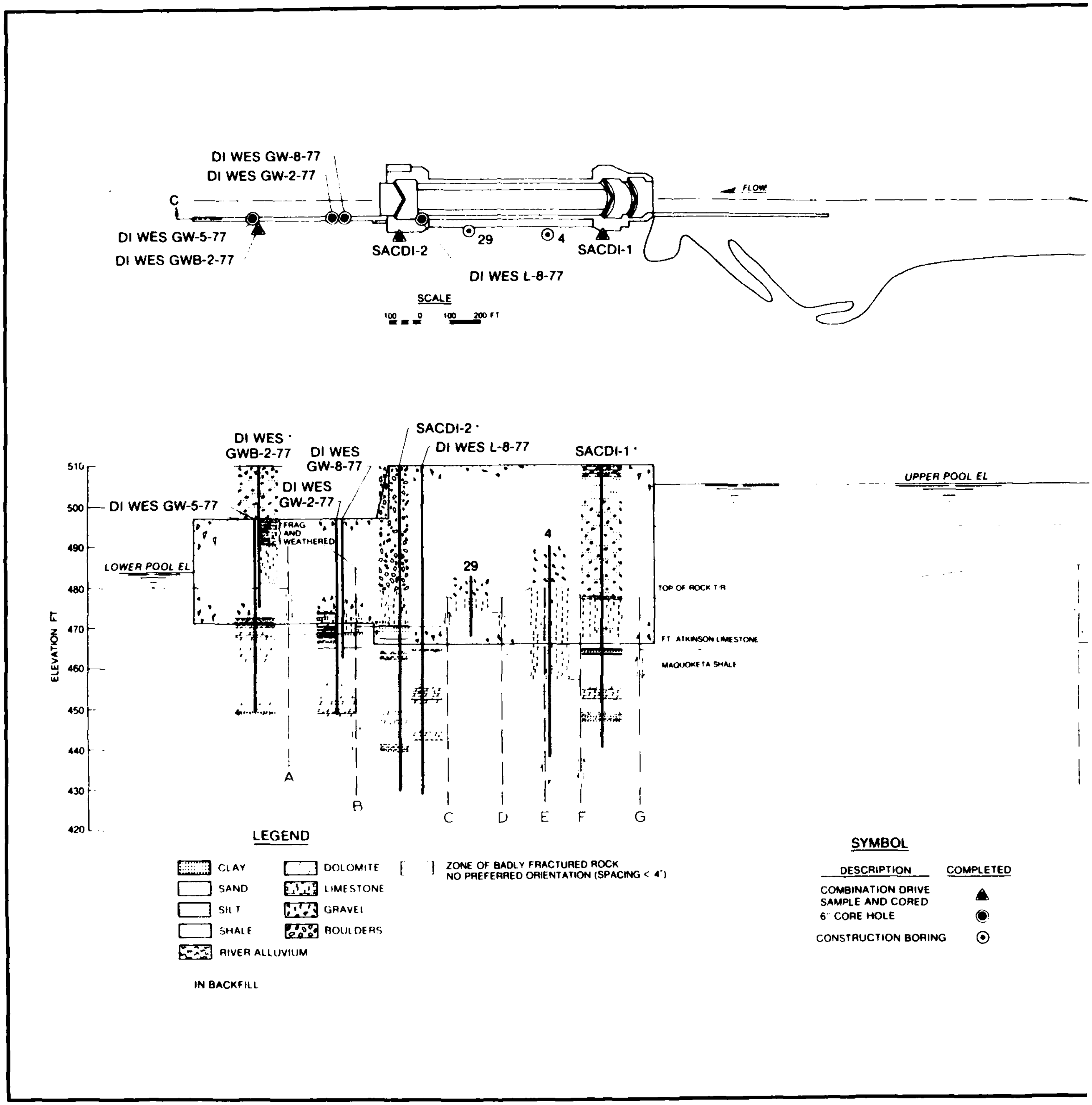
PLATE 13

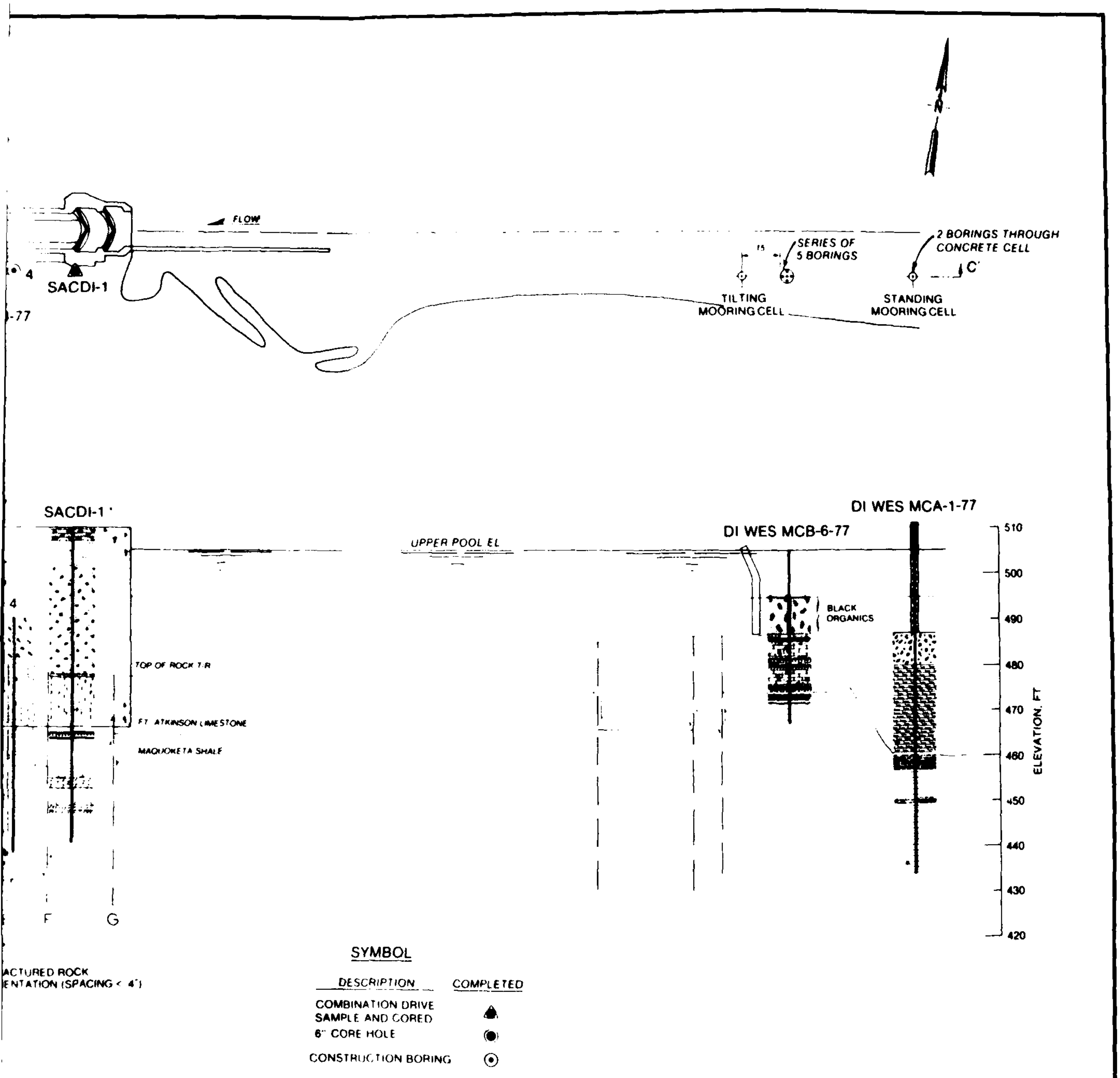
2



DI WES E-1-77
DI WES D-45-79
DI WES E-2-77
SACDI-6
DI WES D-46-79
DI WES D-47-79
FLOW
DI WES D-48-79
DI WES D-9-77
B
SCALE
100 0 100 200 FT
ELEVATION FT
510
500
490
480
470
460
450
440
LOWER POOL EL
TOP OF ROCK
FT ATKINSON LIMESTONE
SLICKENSIDES
MAQUOKETA SHALE
EL 462 75
EL450 75
EL 465 00
EL 453 00
J
K
L
H
LEGEND
CLAY
SAND
SILT
SHALE
DOLOMITE
LIMESTONE
GRAVEL
BOULDERS
CONCRETE
ZONE OF BADLY FRACTURED ROCK NO PREFERRED ORIENTATION (SPACING < 4")
DEPAUPERATE ZONE
* IN BACKFILL
** IN UPSTREAM POOL
SYMBOL
DESCRIPTION
BORING AND PIEZOMETER
COMBINATION DRIVE SAMPLE AND CORED
6" CORE HOLE


FLOW
SACDI-4
SACD1—1
B'
N
DI WES D-47-79
DI WES D-48-79
DI WES D-9-77
DI WES D-49-79
SCALE
100 0 100 200 FT
DI WES D-9-77
SACDI-1
UPPER POOL EL
SACDI-4
DI WES D-47-79
DI WES D-48-79
DI WES D-49-79
ELEVATION, FT
510
500
490
480
470
460
450
440
EL 462 75
EL 465 00
EL 453 00
EL450 75
K
L
H
G
SYMBOL
DESCRIPTION
COMPLETED
BORING AND PIEZOMETER
COMBINATION DRIVE SAMPLE AND CORED
6" CORE HOLE
SHORT DASHED LINES ARE PROBABLE AS BUILT EXTENSION TO CONSTRUCTION DRAWINGS BASED ON ACTUAL BORING DATA
CONCRETE
ZONE OF BADLY FRACTURED ROCK
NO PREFERRED ORIENTATION (SPACING < 4")
DEPAUPERATE ZONE
COMPLIANCE PHASE
DRESDEN ISLAND LOCK AND DAM
ILLINOIS WATERWAY
GEOLOGIC CROSS SECTION
SECTION B-B'


DI WES GW-8-77
DI WES GW-2-77
C
FLOW
DI WES GW-5-77
DI WES GWB-2-77
SACDI-2
29
4
SACDI-1
DI WES L-8-77
SCALE
100 0 100 200 FT
DI WES GWB-2-77
DI WES GW-8-77
DI WES GW-2-77
SACDI-2
DI WES L-8-77
SACDI-1
DI WES GW-5-77
FRAG AND WEATHERED
LOWER POOL EL
UPPER POOL EL
TOP OF ROCK T/R
FT. ATKINSON LIMESTONE
MAQUOKETA SHALE
ELEVATION FT
510
500
490
480
470
460
450
440
430
420
A
B
C
D
E
F
G
LEGEND
CLAY
SAND
SILT
SHALE
RIVER ALLUVIUM
IN BACKFILL
DOLOMITE
LIMESTONE
GRAVEL
BOULDERS
ZONE OF BADLY FRACTURED ROCK NO PREFERRED ORIENTATION (SPACING < 4")
SYMBOL
DESCRIPTION
COMPLETED
COMBINATION DRIVE SAMPLE AND CORED
6" CORE HOLE
CONSTRUCTION BORING



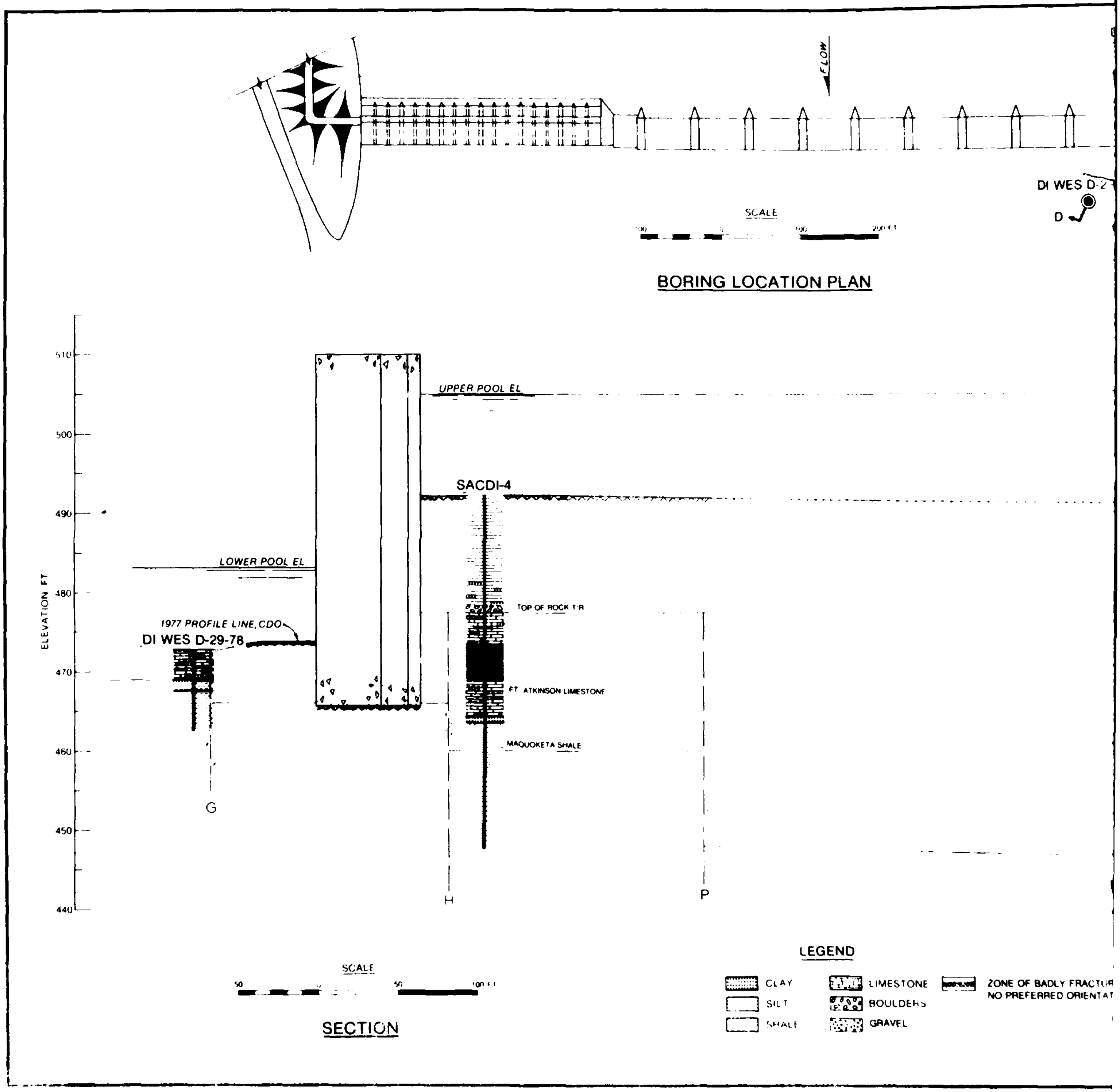
SCOUR DETECTION
DRESDEN ISLAND LOCK AND DAM
ILLINOIS WATERWAY

GEOLOGIC CROSS SECTION I

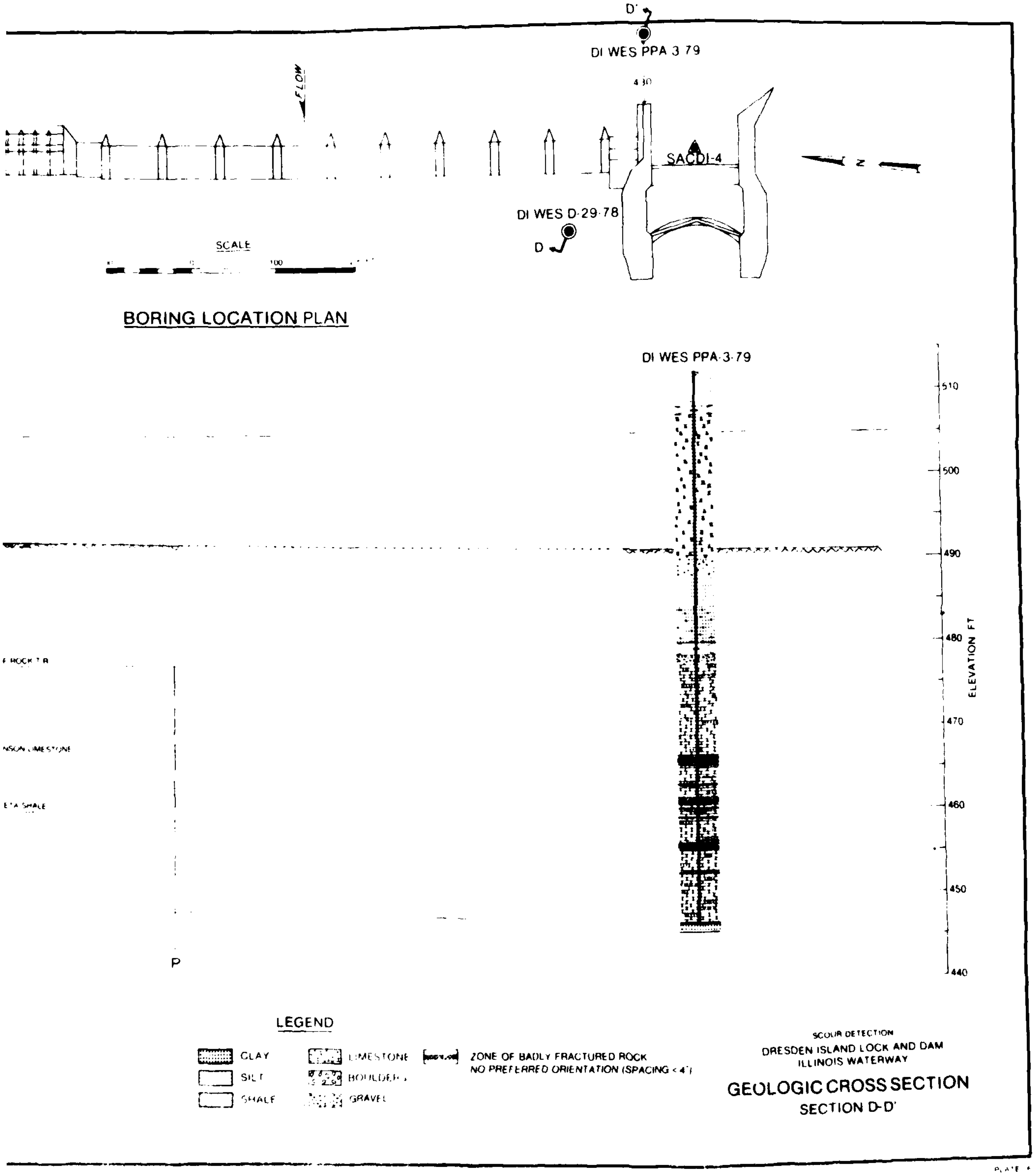
SECTION C-C'

PLATE 15

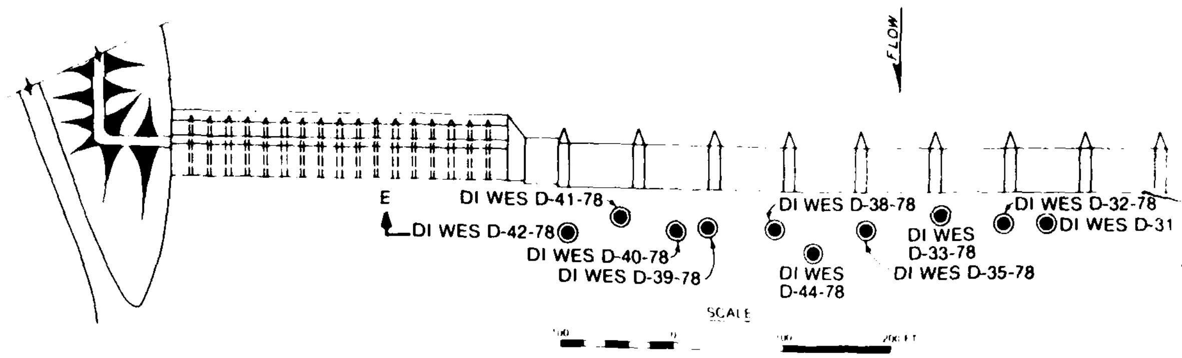
2



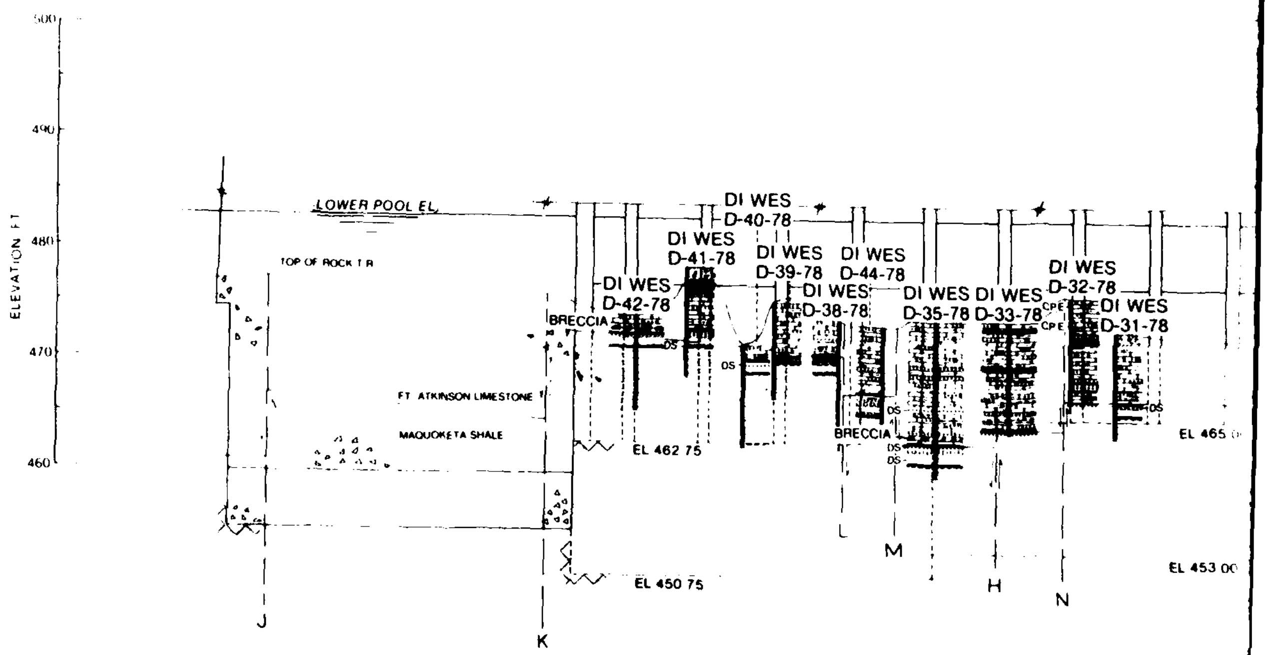
FLOW
DI WES D-2
D
SCALE
BORING LOCATION PLAN
ELEVATION FT
510
500
490
480
470
460
450
440
UPPER POOL EL
SACDI-4
LOWER POOL EL
TOP OF ROCK T/R
1977 PROFILE LINE, CDO
DI WES D-29-78
FT ATKINSON LIMESTONE
MAQUOKETA SHALE
G
H
P
SCALE
SECTION
LEGEND
CLAY
SILT
SHALE
LIMESTONE
BOULDERS
GRAVEL
ZONE OF BADLY FRACTUR
NO PREFERRED ORIENTAT



D'
DI WES PPA 3 79
FLOW
SACDI-4
N
DI WES D-29-78
D
SCALE
100
BORING LOCATION PLAN
DI WES PPA-3-79
510
500
490
480
470
460
450
440
ELEVATION FT
P
LEGEND
CLAY
SILT
SHALE
LIMESTONE
BOULDERS
GRAVEL
ZONE OF BADLY FRACTURED ROCK
NO PREFERRED ORIENTATION (SPACING < 4")
SCOUR DETECTION
DRESDEN ISLAND LOCK AND DAM
ILLINOIS WATERWAY
GEOLOGIC CROSS SECTION
SECTION D-D'



BORING LOCATION PLAN



SECTION

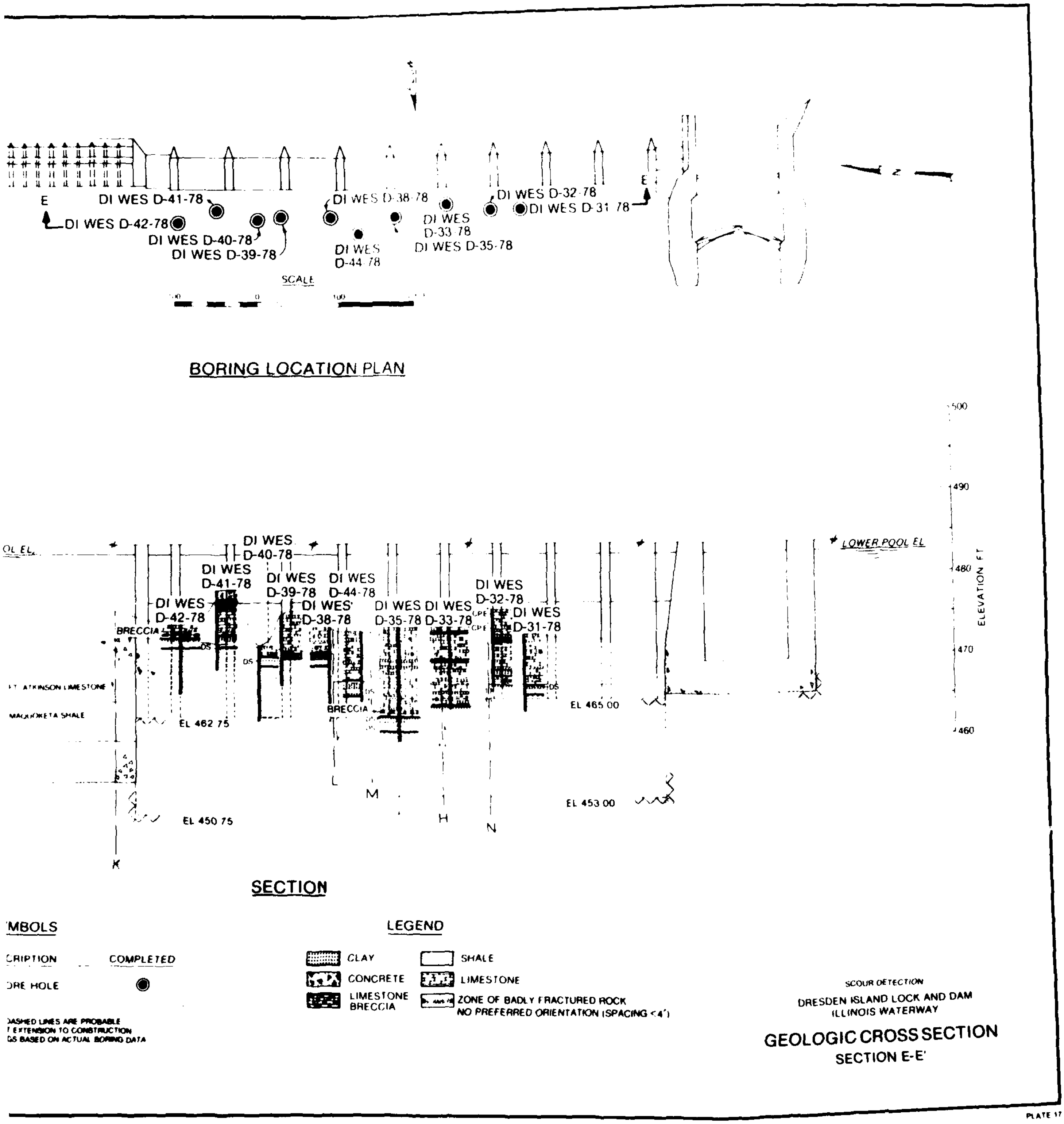
SYMBOLS

DESCRIPTION	COMPLETED
6" CORE HOLE	◉

SHORT DASHED LINES ARE PROBABLE AS BUILT EXTENSION TO CONSTRUCTION DRAWINGS BASED ON ACTUAL BORING DATA

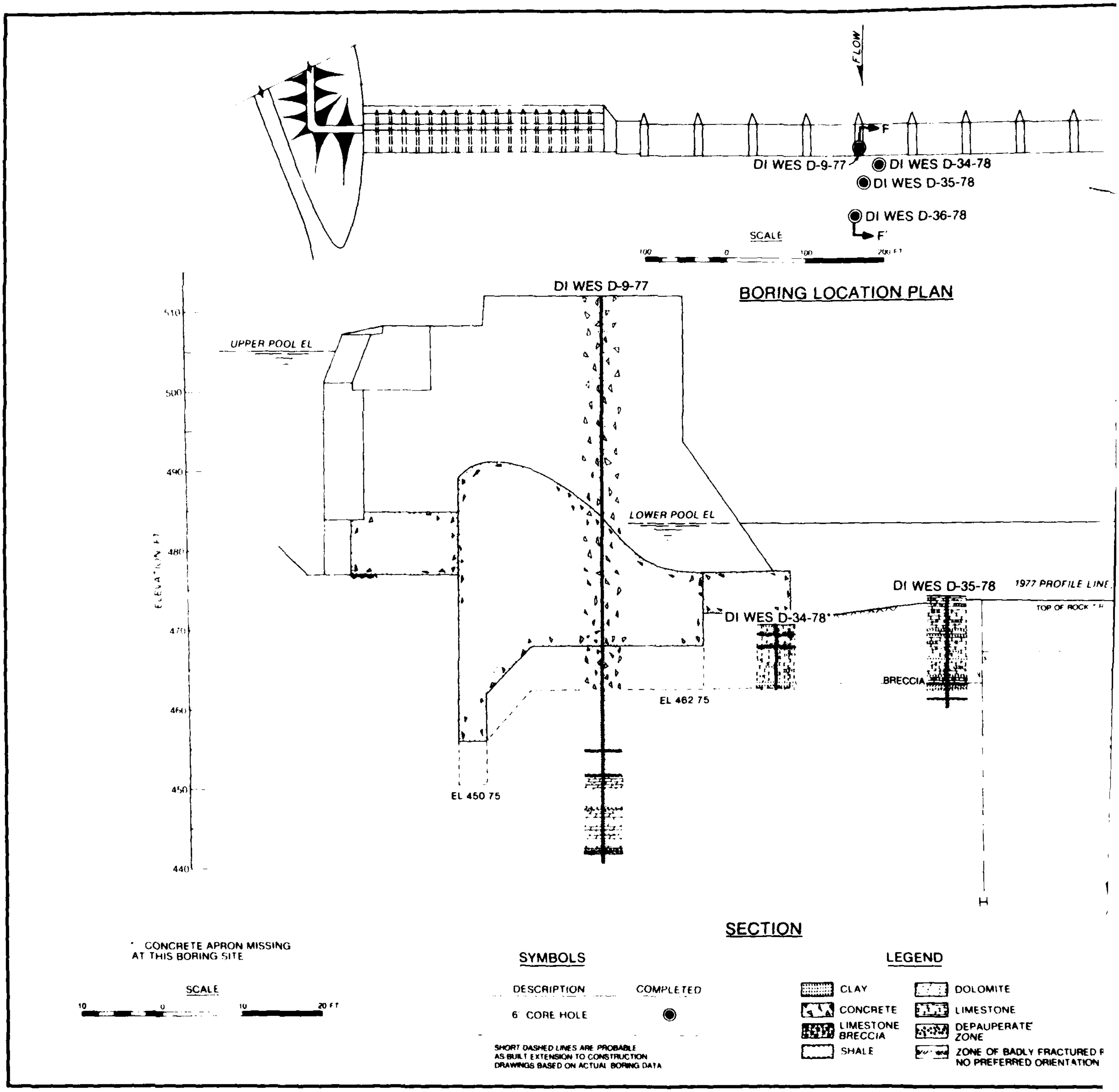
LEGEND

- CLAY
- CONCRETE
- LIMESTONE BRECCIA
- SHALE
- LIMESTONE
- ZONE OF BADLY FRACTURED ROCK NO PREFERRED ORIENTATION (SPA



BORING LOCATION PLAN
DI WES D-41-78
DI WES D-42-78
DI WES D-40-78
DI WES D-39-78
DI WES D-38-78
DI WES D-44-78
DI WES D-33-78
DI WES D-35-78
DI WES D-32-78
DI WES D-31-78
SCALE
SECTION
LOWER POOL EL
ELEVATION FT
BRECCIA
FT. ATKINSON LIMESTONE
MAQUOKETA SHALE
EL 462 75
EL 450 75
EL 465 00
EL 453 00
'MBOLS
CRIPTION
COMPLETED
ORE HOLE
LEGEND
CLAY
CONCRETE
LIMESTONE BRECCIA
SHALE
LIMESTONE
ZONE OF BADLY FRACTURED ROCK NO PREFERRED ORIENTATION (SPACING <4')
SCOUR DETECTION
DRESDEN ISLAND LOCK AND DAM
ILLINOIS WATERWAY
GEOLOGIC CROSS SECTION
SECTION E-E'

PLATE 17



FLOW
F
DI WES D-9-77
DI WES D-34-78
DI WES D-35-78
DI WES D-36-78
F'
SCALE
100 0 100 200 FT
BORING LOCATION PLAN
DI WES D-9-77
ELEVATION FT
510
500
490
480
470
460
450
440
UPPER POOL EL
LOWER POOL EL
DI WES D-35-78
1977 PROFILE LINE
TOP OF ROCK
DI WES D-34-78*
BRECCIA
EL 462 75
EL 450 75
H
SECTION
* CONCRETE APRON MISSING AT THIS BORING SITE
SCALE
10 0 10 20 FT
SYMBOLS
DESCRIPTION
COMPLETED
6 CORE HOLE
SHORT DASHED LINES ARE PROBABLE AS BUILT EXTENSION TO CONSTRUCTION DRAWINGS BASED ON ACTUAL BORING DATA
LEGEND
CLAY
CONCRETE
LIMESTONE BRECCIA
SHALE
DOLOMITE
LIMESTONE
DEPAUPERATE ZONE
ZONE OF BADLY FRACTURED NO PREFERRED ORIENTATION

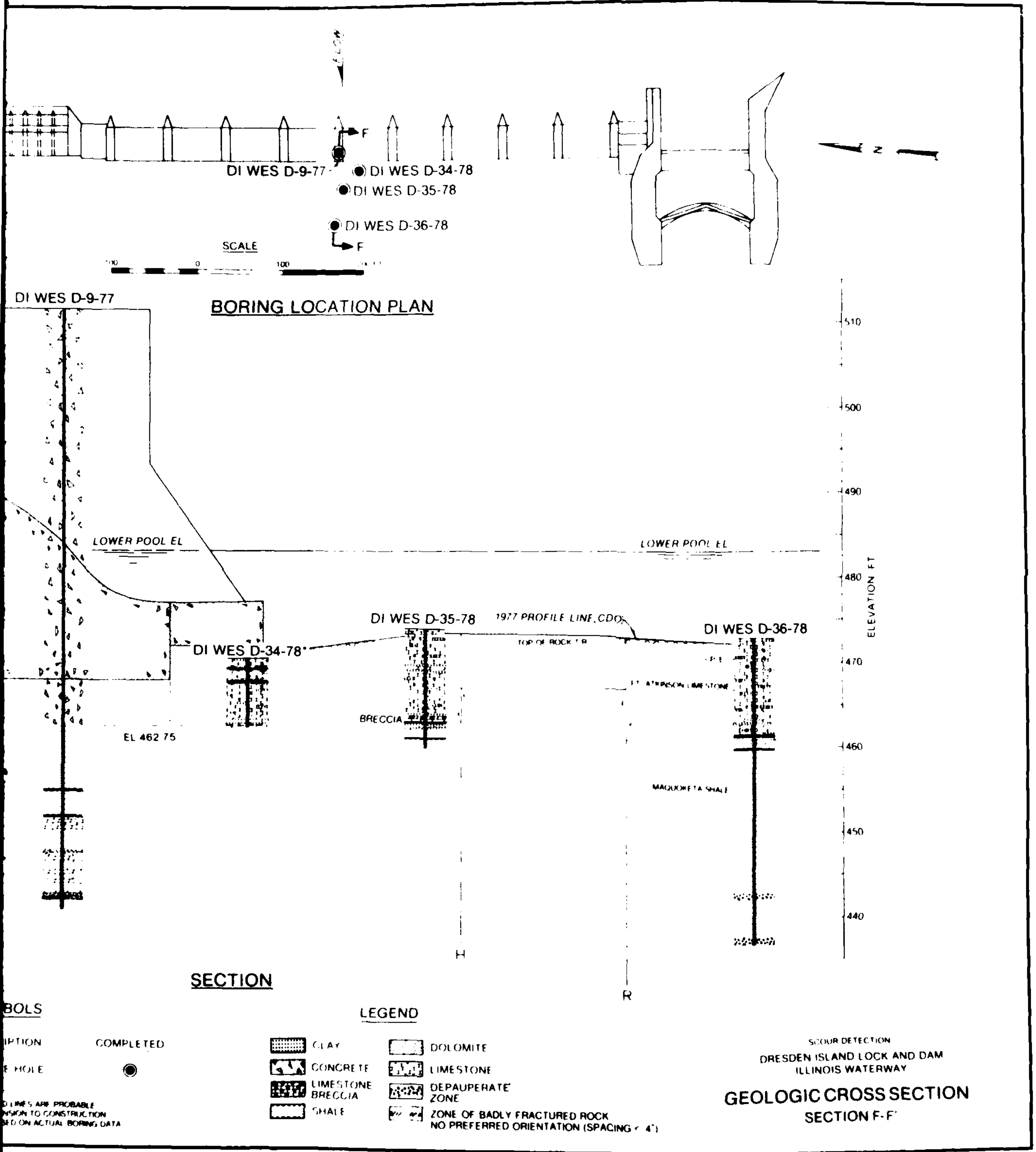
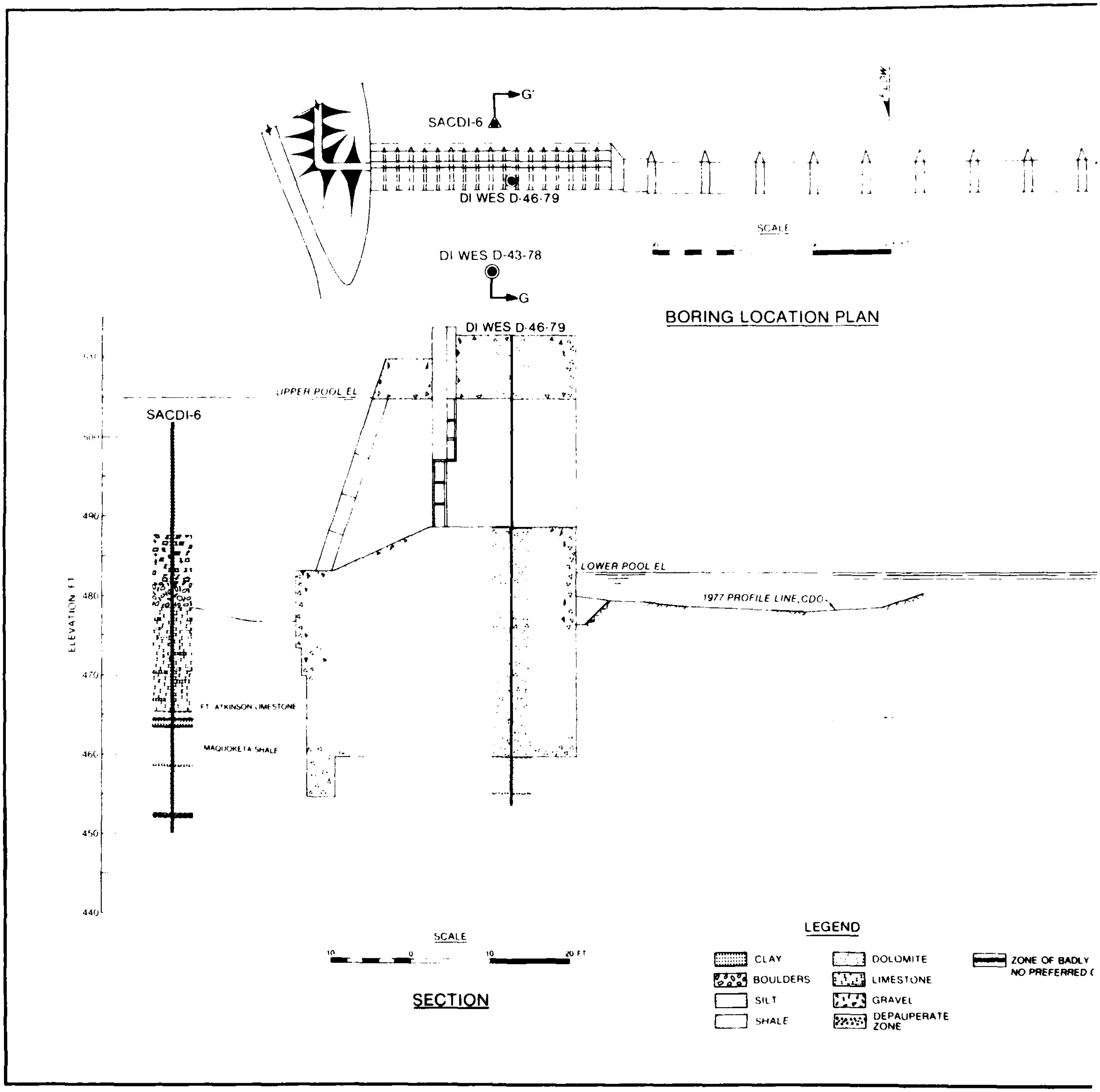

FLOW
F
DI WES D-9-77
DI WES D-34-78
DI WES D-35-78
DI WES D-36-78
F
SCALE
100
0
100
BORING LOCATION PLAN
DI WES D-9-77
LOWER POOL EL
LOWER POOL EL
DI WES D-35-78
1977 PROFILE LINE, CDO
TOP OF ROCK
DI WES D-36-78
DI WES D-34-78
FT. ATKINSON LIMESTONE
BRECCIA
EL 462 75
MAQUOKETA SHALE
510
500
490
480
470
460
450
440
ELEVATION FT
H
R
SECTION
BOLS
LEGEND
IPTION
COMPLETED
E HOLE
D LINES ARE PROBABLE
NSION TO CONSTRUCTION
ED ON ACTUAL BORING DATA
CLAY
CONCRETE
LIMESTONE BRECCIA
SHALE
DOLOMITE
LIMESTONE
DEPAUPERATE ZONE
ZONE OF BADLY FRACTURED ROCK NO PREFERRED ORIENTATION (SPACING < 4")
SCOUR DETECTION
DRESDEN ISLAND LOCK AND DAM
ILLINOIS WATERWAY
GEOLOGIC CROSS SECTION
SECTION F-F'

PLATE 18


SACDI-6
G'
DI WES D-46-79
DI WES D-43-78
G
SCALE
BORING LOCATION PLAN
DI WES D-46-79
UPPER POOL EL
SACDI-6
ELEVATION FT
490
480
470
460
450
440
FT. ATKINSON LIMESTONE
MAQUOKETA SHALE
LOWER POOL EL
1977 PROFILE LINE, CDO
SCALE
10
0
10
20 FT
SECTION
LEGEND
CLAY
BOULDERS
SILT
SHALE
DOLOMITE
LIMESTONE
GRAVEL
DEPAUPERATE ZONE
ZONE OF BADLY
NO PREFERRED

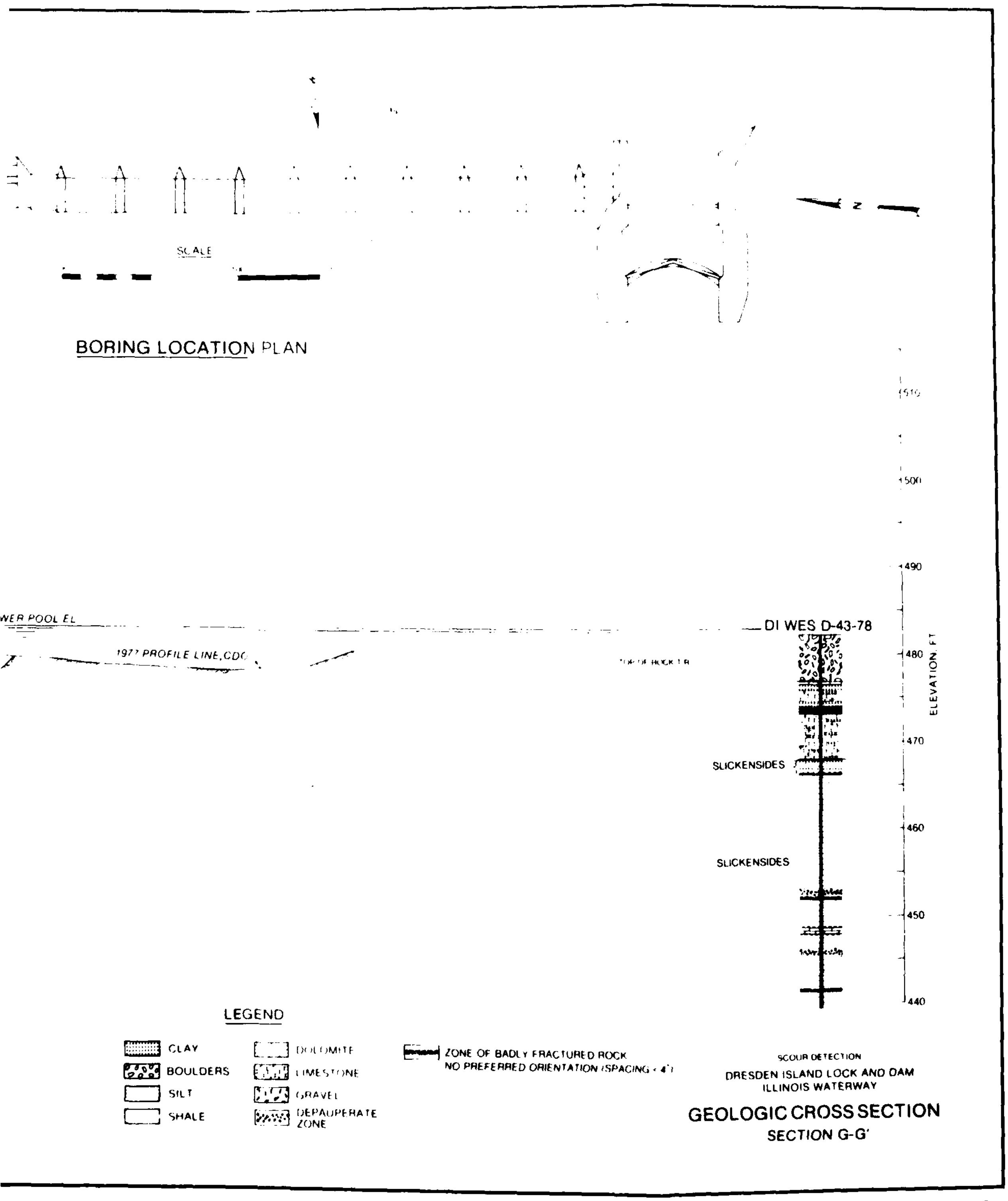
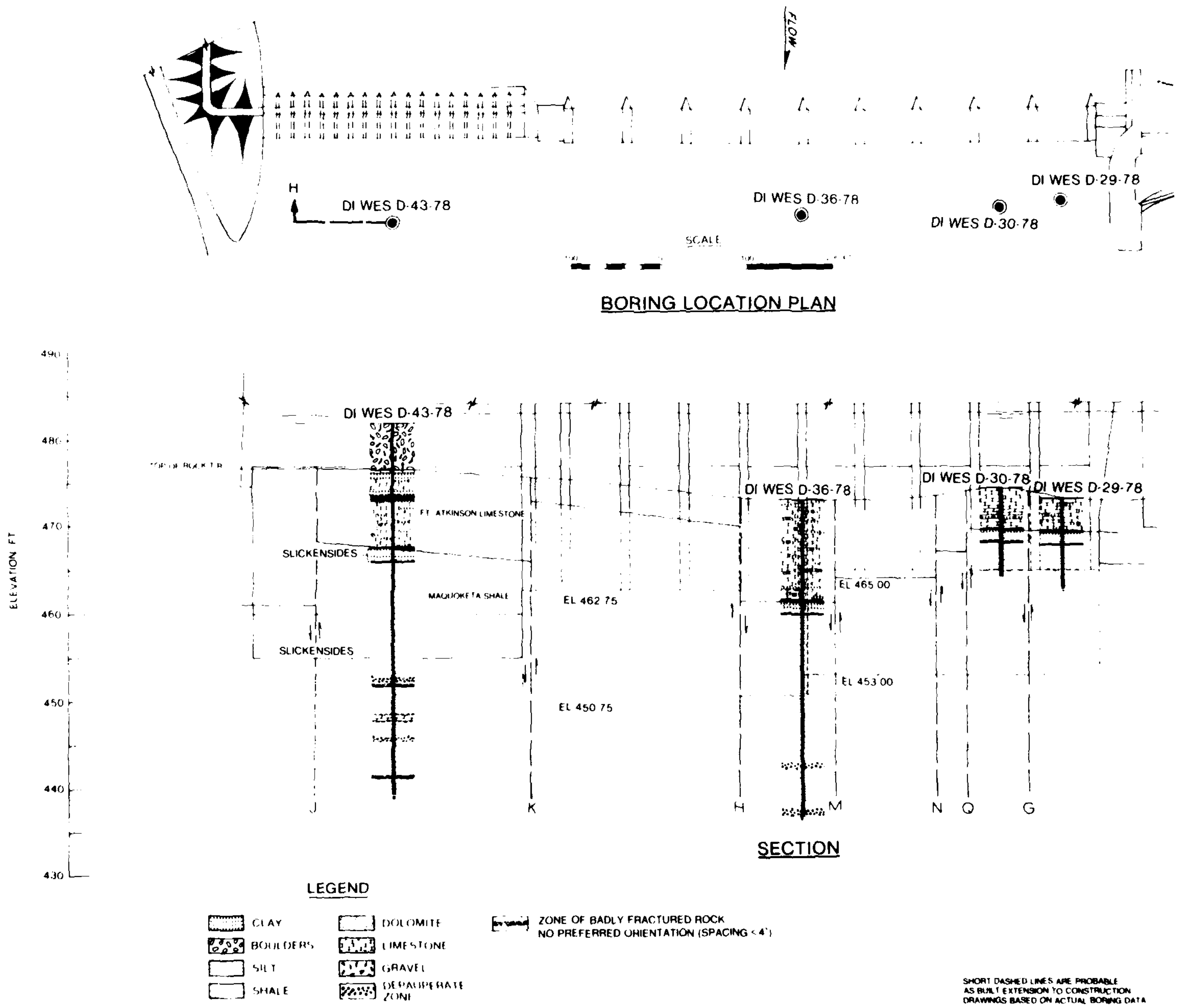
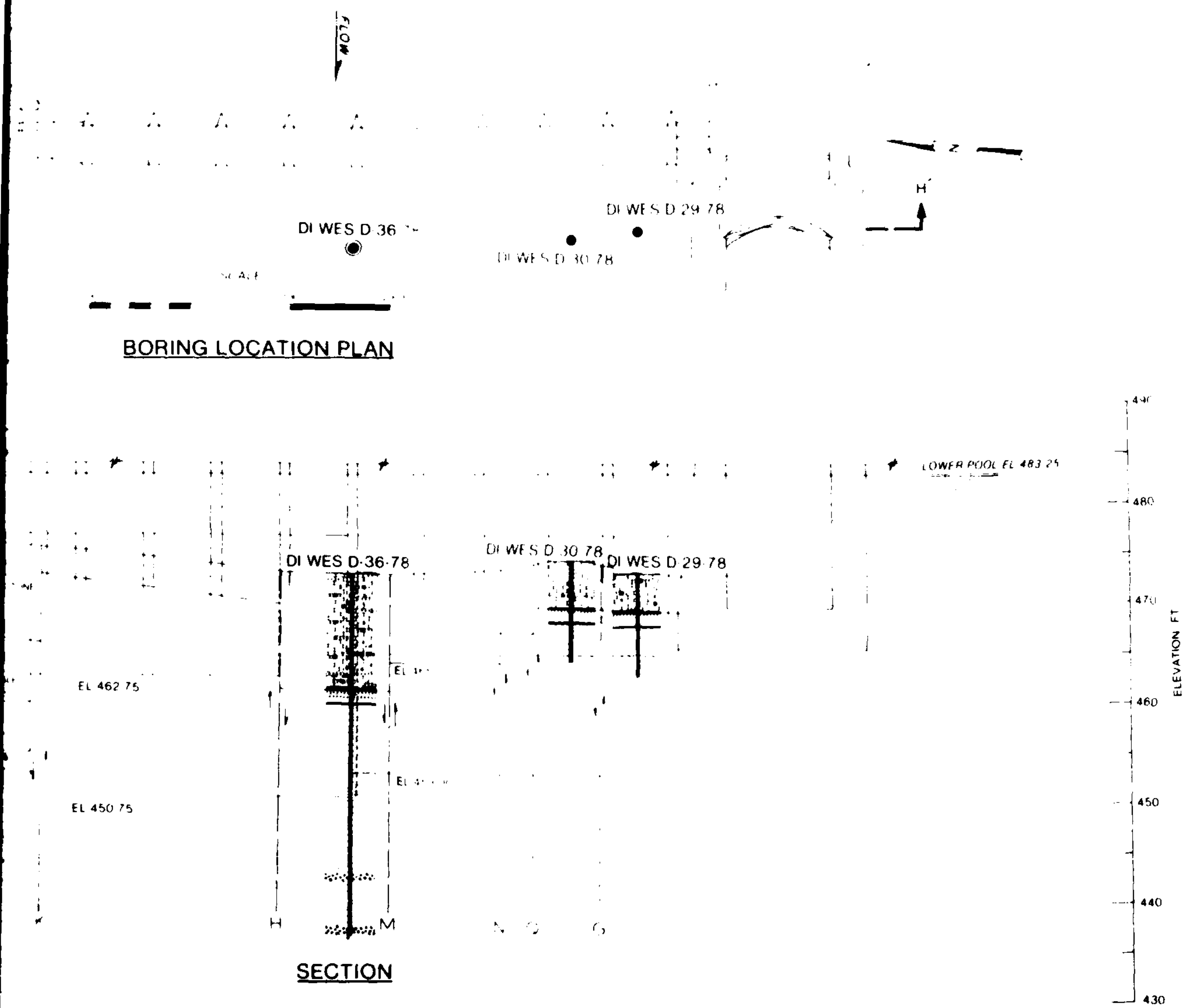

BORING LOCATION PLAN
SCALE
DI WES D-43-78
ELEVATION, FT
490
480
470
460
450
440
SLICKENSIDES
SLICKENSIDES
LEGEND
CLAY
BOULDERS
SILT
SHALE
DOLOMITE
LIMESTONE
GRAVEL
DEPAUPERATE ZONE
ZONE OF BADLY FRACTURED ROCK
NO PREFERRED ORIENTATION (SPACING < 4")
SCOUR DETECTION
DRESDEN ISLAND LOCK AND DAM
ILLINOIS WATERWAY
GEOLOGIC CROSS SECTION
SECTION G-G'

PLATE 19a


FLOW
DI WES D-43-78
DI WES D-36-78
DI WES D-30-78
DI WES D-29-78
H
SCALE
BORING LOCATION PLAN
490
480
470
460
450
440
430
ELEVATION FT
DI WES D-43-78
FT. ATKINSON LIMESTONE
SLICKENSIDES
MAQUOKETA SHALE
SLICKENSIDES
EL 462 75
EL 450 75
DI WES D-36-78
EL 465 00
EL 453 00
DI WES D-30-78
DI WES D-29-78
J
K
H
M
N
Q
G
SECTION
LEGEND
CLAY
BOULDERS
SILT
SHALE
DOLOMITE
LIMESTONE
GRAVEL
DEPAUPERATE ZONE
ZONE OF BADLY FRACTURED ROCK
NO PREFERRED ORIENTATION (SPACING < 4')
SHORT DASHED LINES ARE PROBABLE
AS BUILT EXTENSION TO CONSTRUCTION
DRAWINGS BASED ON ACTUAL BORING DATA



ZONE OF BADLY FRACTURED ROCK
NO PREFERRED ORIENTATION (SPACING < 4")

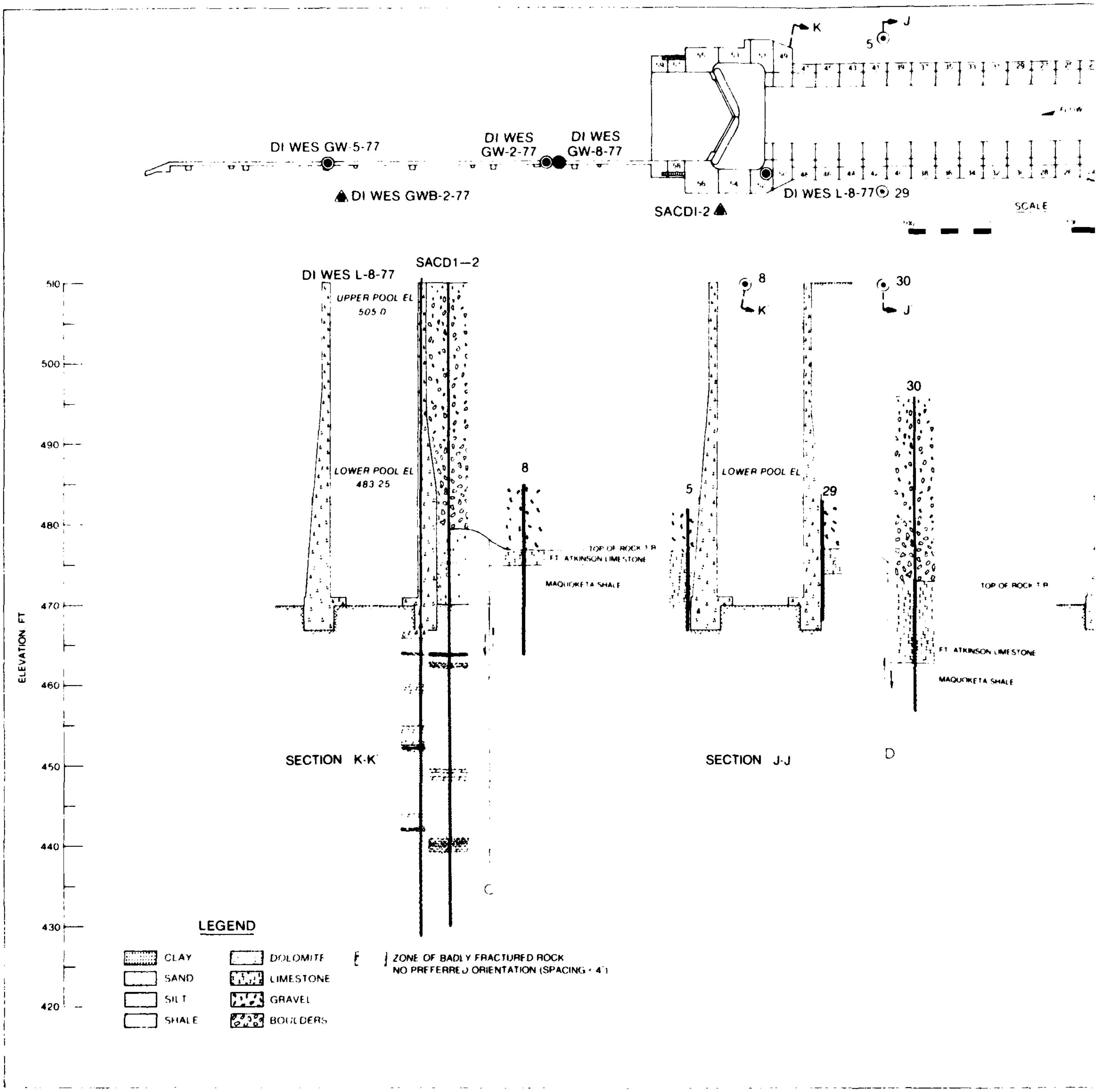
SHORT DASHED LINES ARE PROBABLE
AS-BUILT EXTENSION TO CONSTRUCTION
DRAWINGS BASED ON ACTUAL BORING DATA

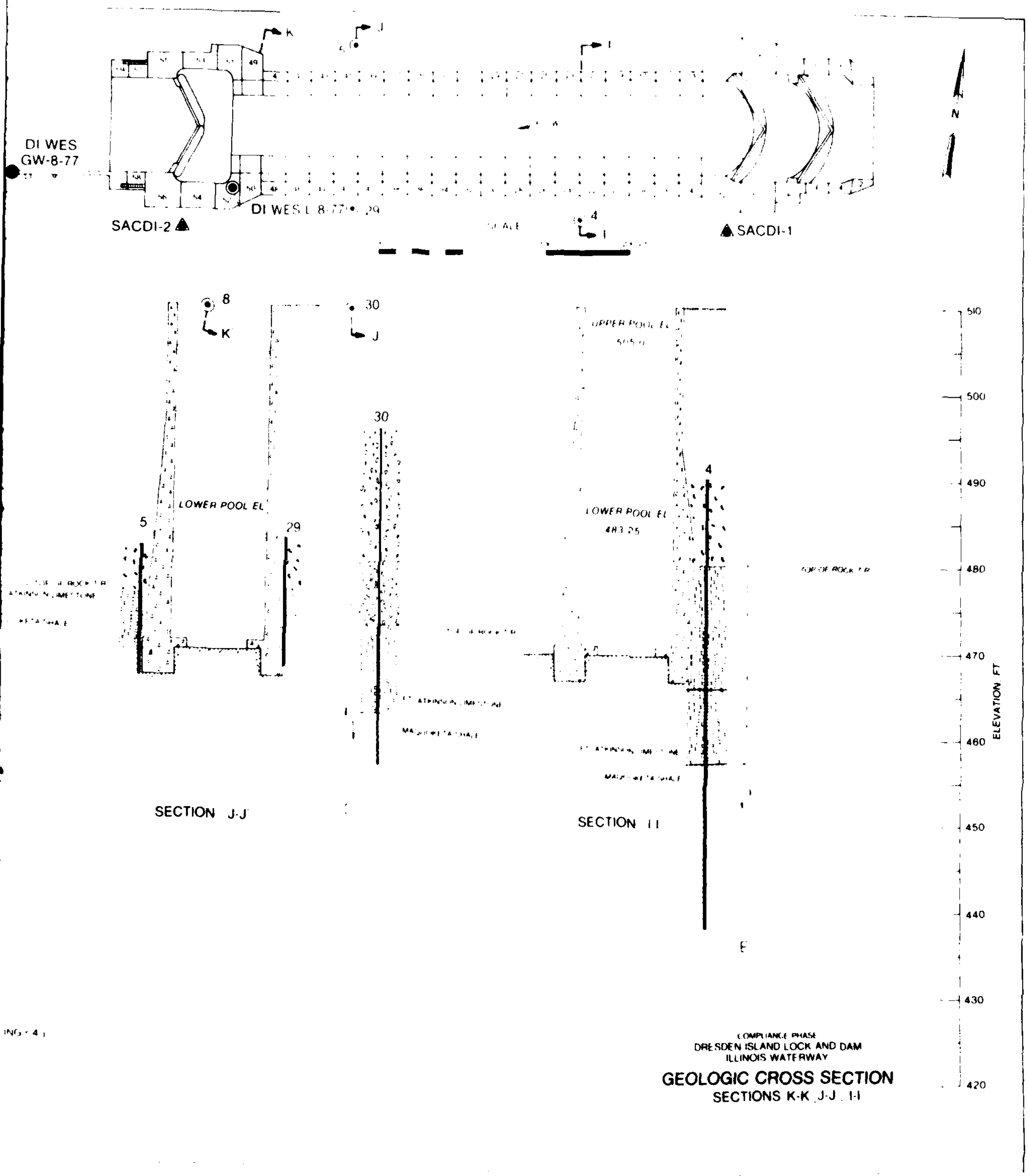
SCOUR DETECTION
DRESDEN ISLAND LOCK AND DAM
ILLINOIS WATERWAY

GEOLOGIC CROSS SECTION

SECTION H-H'

PLATE 19b

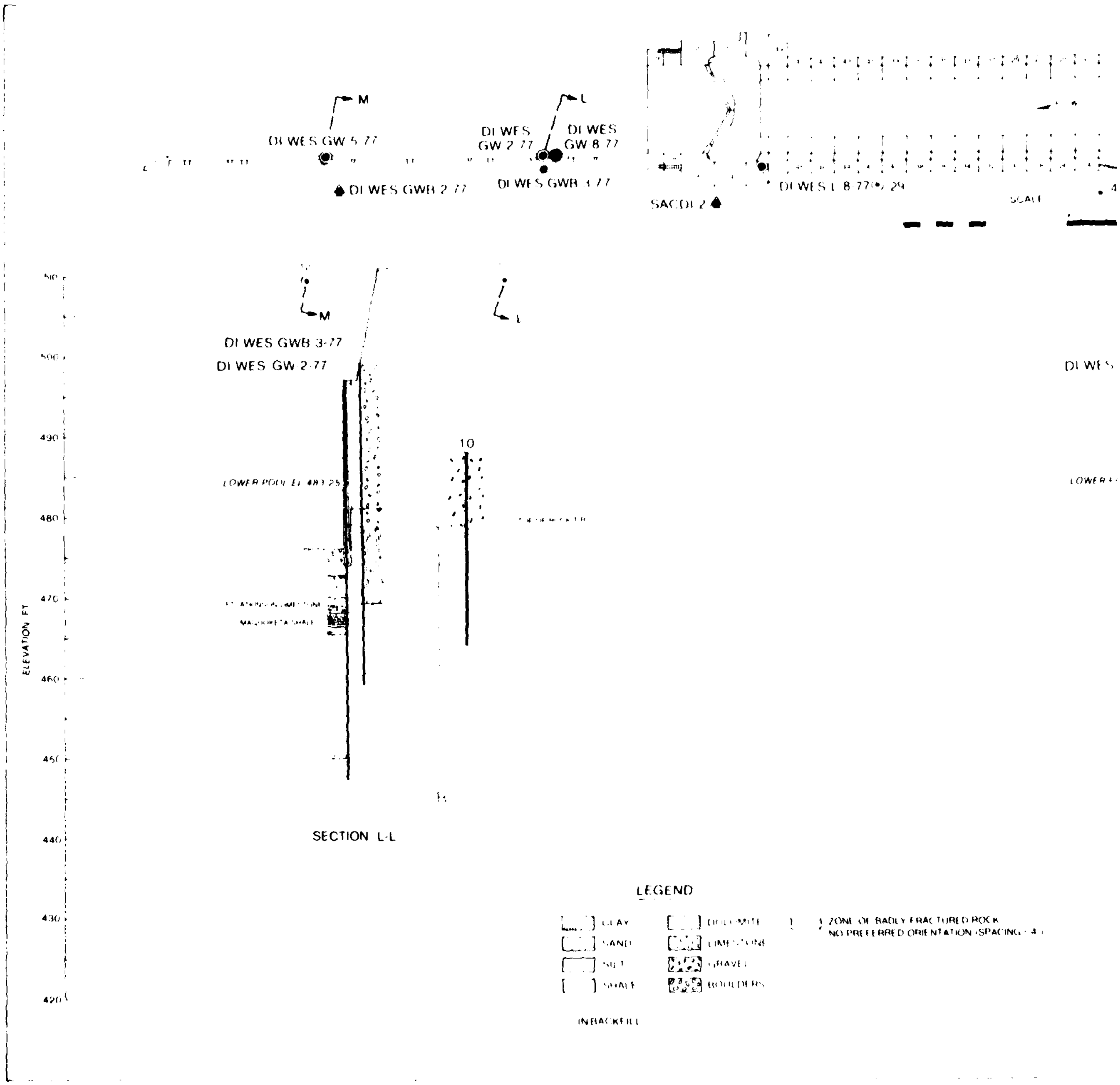

DI WES GW-5-77
DI WES GW-2-77
DI WES GW-8-77
DI WES GWB-2-77
SACDI-2
DI WES L-8-77 29
K
J
5
SCALE
DI WES L-8-77
SACD1—2
UPPER POOL EL 505.0
LOWER POOL EL 483.25
8
TOP OF ROCK
FT. ATKINSON LIMESTONE
MAQUOKETA SHALE
5
29
30
LOWER POOL EL
ELEVATION FT
510
500
490
480
470
460
450
440
430
420
SECTION K-K
SECTION J-J
D
C
LEGEND
CLAY
SAND
SILT
SHALE
DOLOMITE
LIMESTONE
GRAVEL
BOULDERS
ZONE OF BADLY FRACTURED ROCK NO PREFERRED ORIENTATION (SPACING < 4')



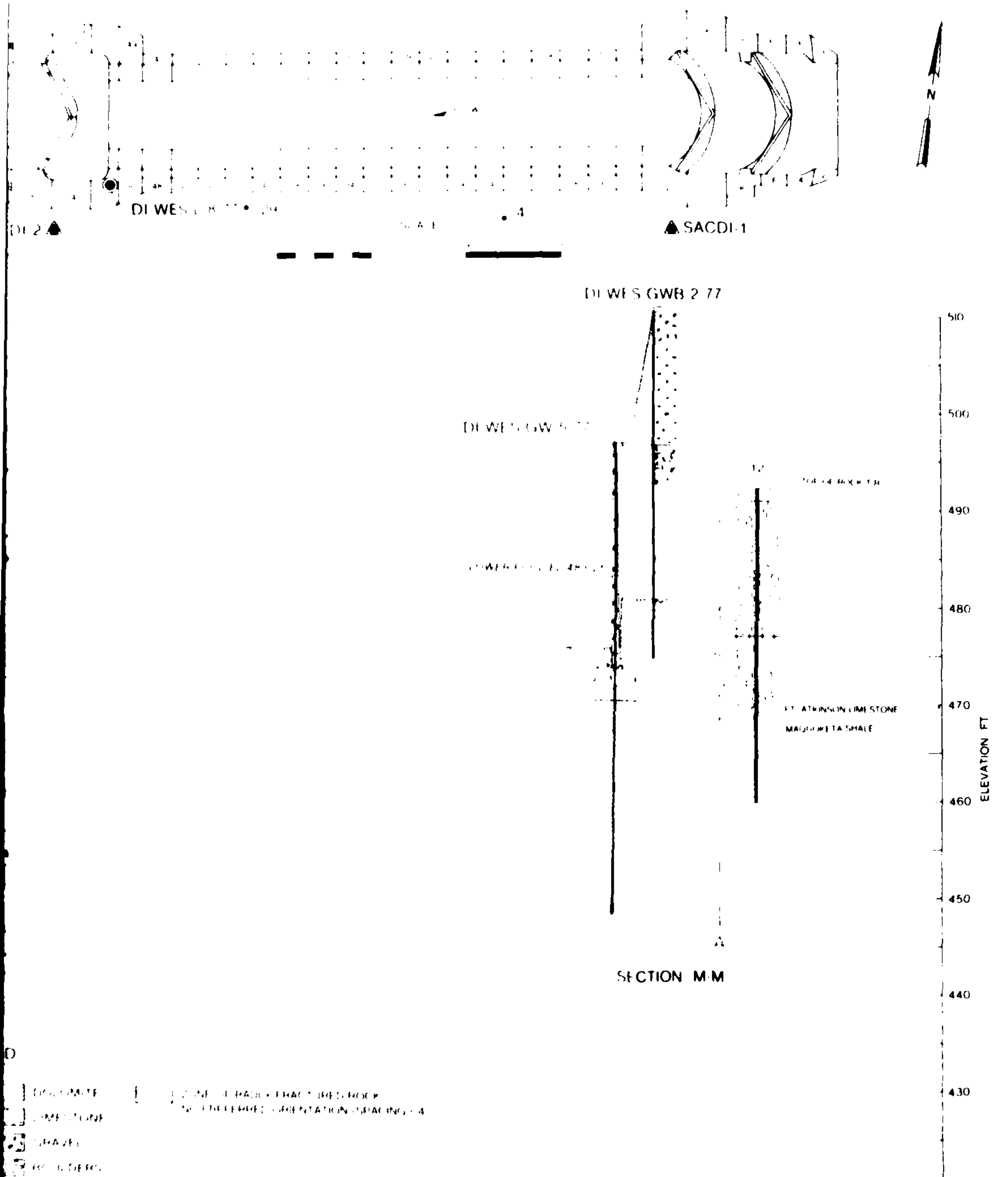
COMPLIANCE PHASE
DRESDEN ISLAND LOCK AND DAM
ILLINOIS WATERWAY

GEOLOGIC CROSS SECTION
SECTIONS K-K, J-J, I-I

PLATE 19c


DI WES GW 5-77
DI WES GWB 2-77
DI WES GW 2-77
DI WES GW 8-77
DI WES GWB 3-77
SACDL 2
SCALE
DI WES GWB 3-77
DI WES GW 2-77
LOWER POOL EL 483.25
ELEVATION, FT
510
500
490
480
470
460
450
440
430
420
10
DI WES
LEGEND
CLAY
SAND
SILT
SHALE
DOLOMITE
LIMESTONE
GRAVEL
BOULDERS
ZONE OF BADLY FRACTURED ROCK
NO PREFERRED ORIENTATION
IN BACKFILL

SECTION L-L



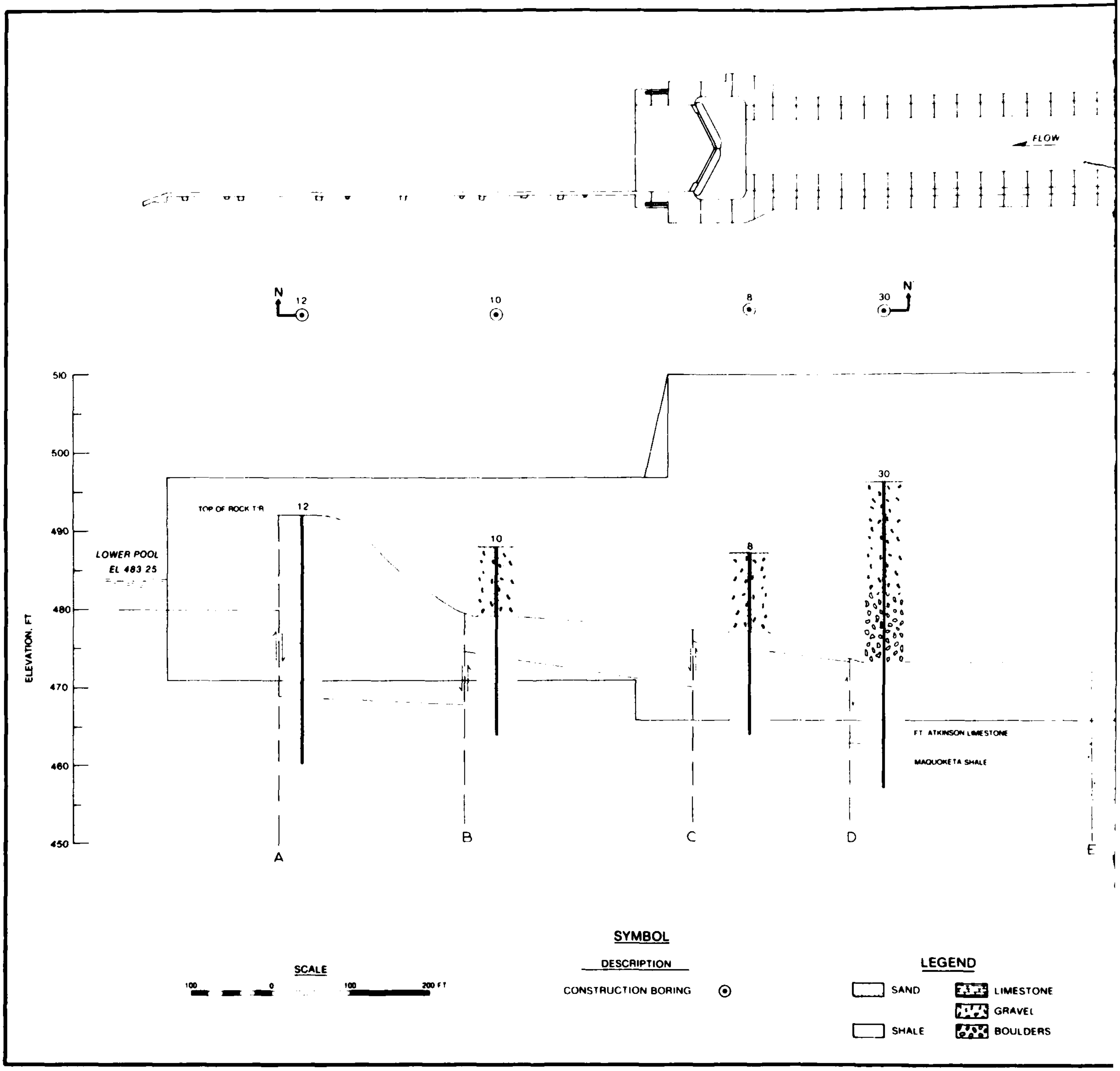
COMPLIANCE PHASE
DRESDEN ISLAND LOCK AND DAM
ILLINOIS WATERWAY

GEOLOGIC CROSS SECTION

SECTION L-L M-M'

PLATE 19d

2



FLOW
N
12
10
8
30
N
ELEVATION, FT
510
500
490
480
470
460
450
LOWER POOL
EL 483 25
TOP OF ROCK T/R
12
10
8
30
FT ATKINSON LIMESTONE
MAQUOKETA SHALE
A
B
C
D
E
SCALE
100
0
100
200 FT
SYMBOL
DESCRIPTION
CONSTRUCTION BORING
LEGEND
SAND
SHALE
LIMESTONE
GRAVEL
BOULDERS

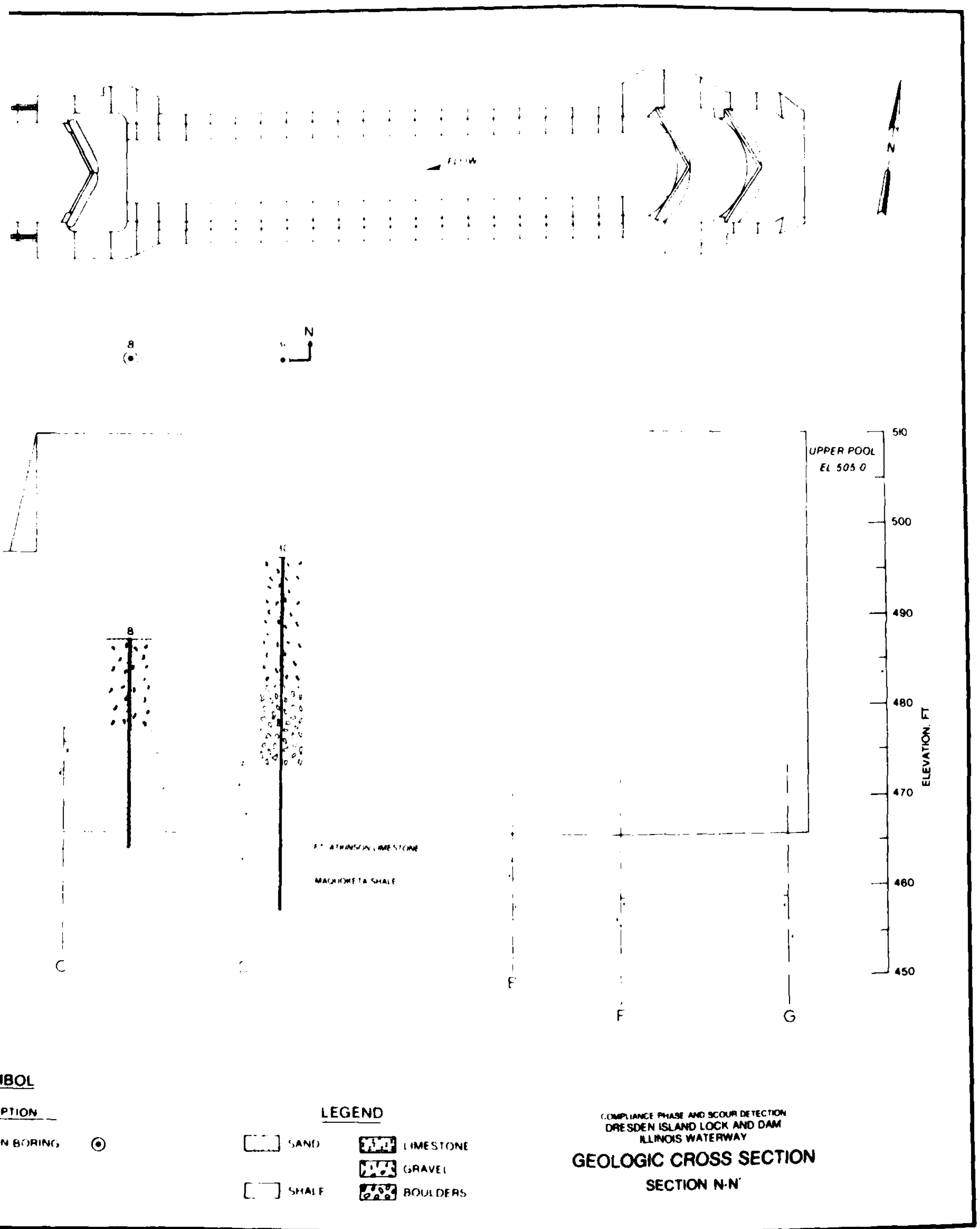
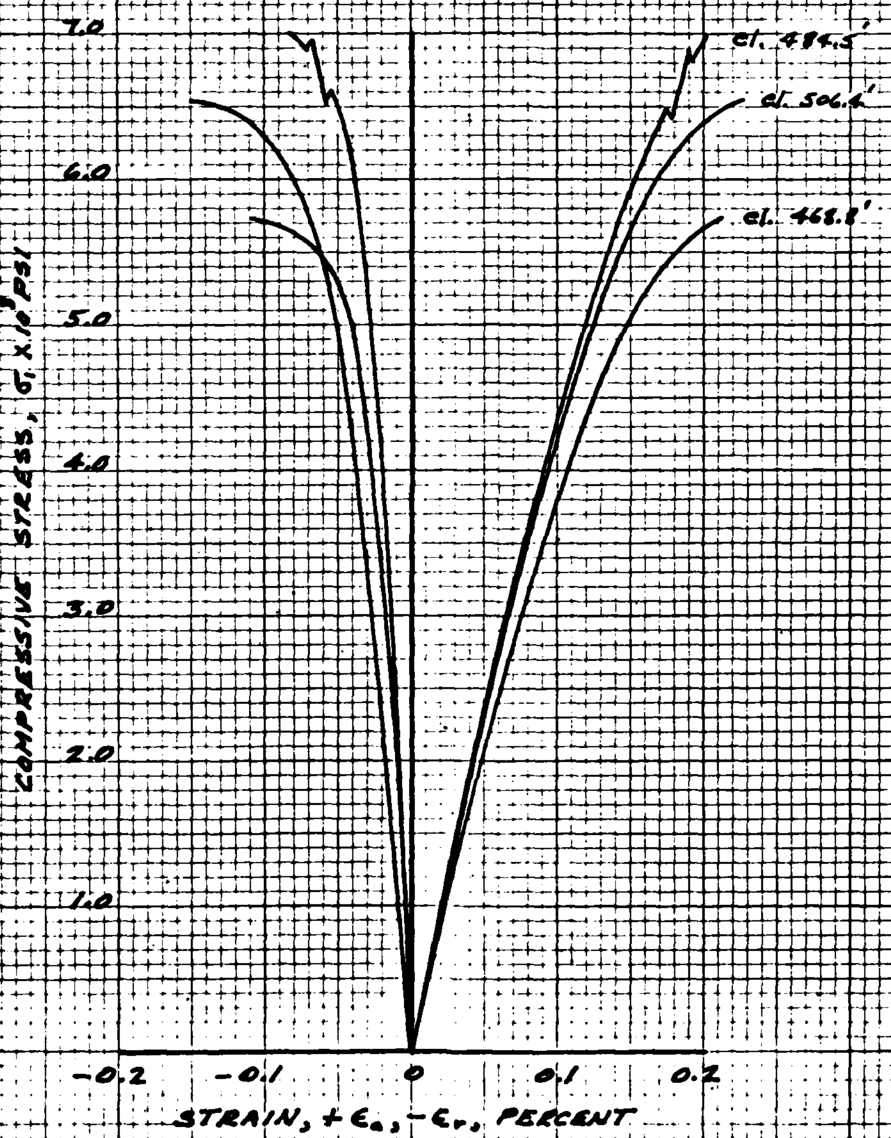

FLOW
N
UPPER POOL
EL 505.0
510
500
490
480
470
460
450
ELEVATION, FT
FT. ATKINSON LIMESTONE
MAQUOKETA SHALE
C
F
G
BOL
PTION
N BORING
LEGEND
SAND
SHALE
LIMESTONE
GRAVEL
BOULDERS
COMPLIANCE PHASE AND SCOUR DETECTION
DRESDEN ISLAND LOCK AND DAM
ILLINOIS WATERWAY
GEOLOGIC CROSS SECTION
SECTION N-N'

PLATE 19e

2

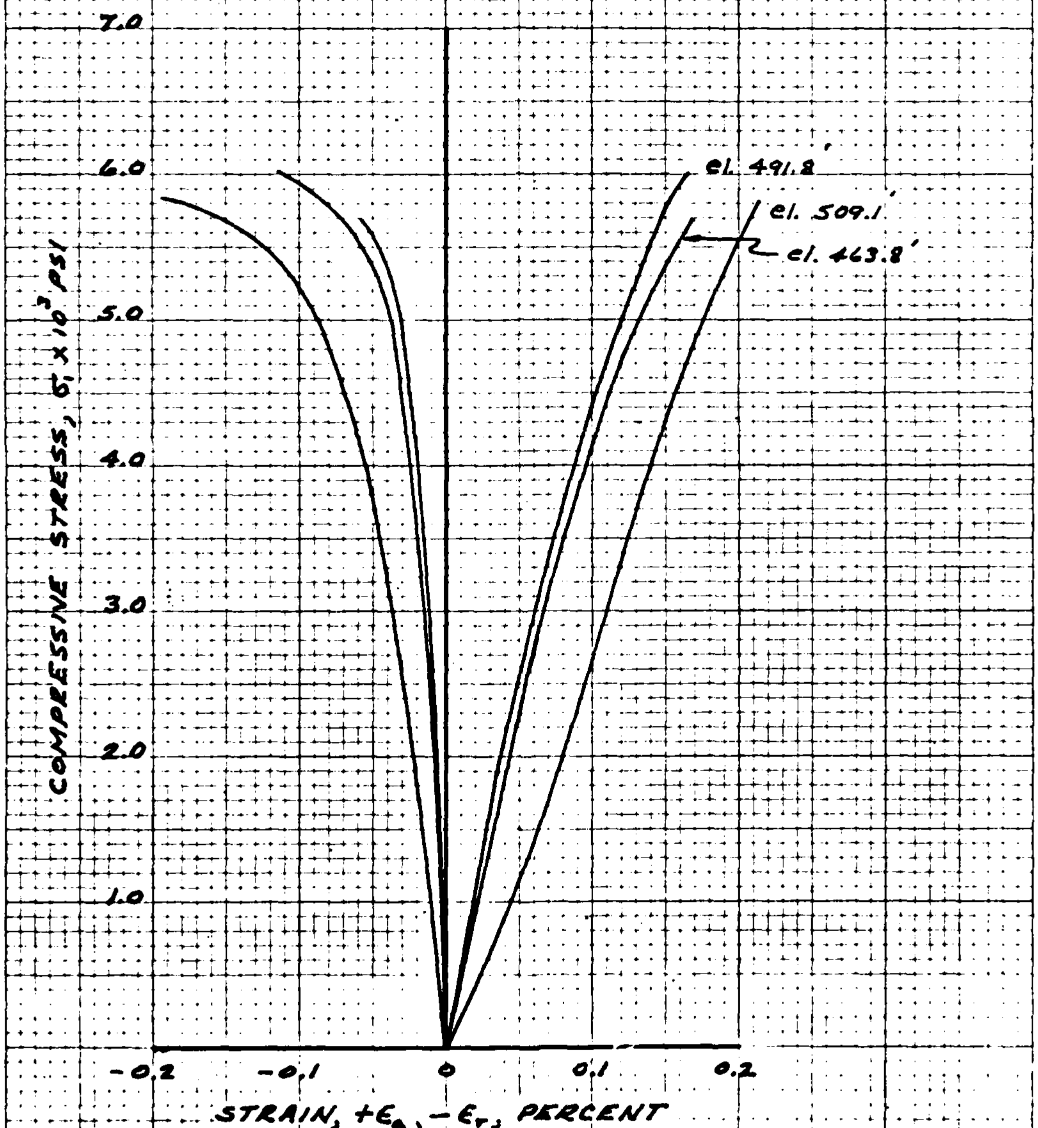
Hole No.	Elev. FT	σ_1 PSI	Modulus $E \times 10^6$, psi	Poisson's Ratio	Moisture Percent
L 8	506.4	6570	4.65	0.38	6.0
L 8	484.5	7230	4.49	0.20	7.2
L 8	468.8	5740	4.30	0.22	8.1



STRESS-STRAIN RELATION FOR CONCRETE, DI LOCK AND DAM

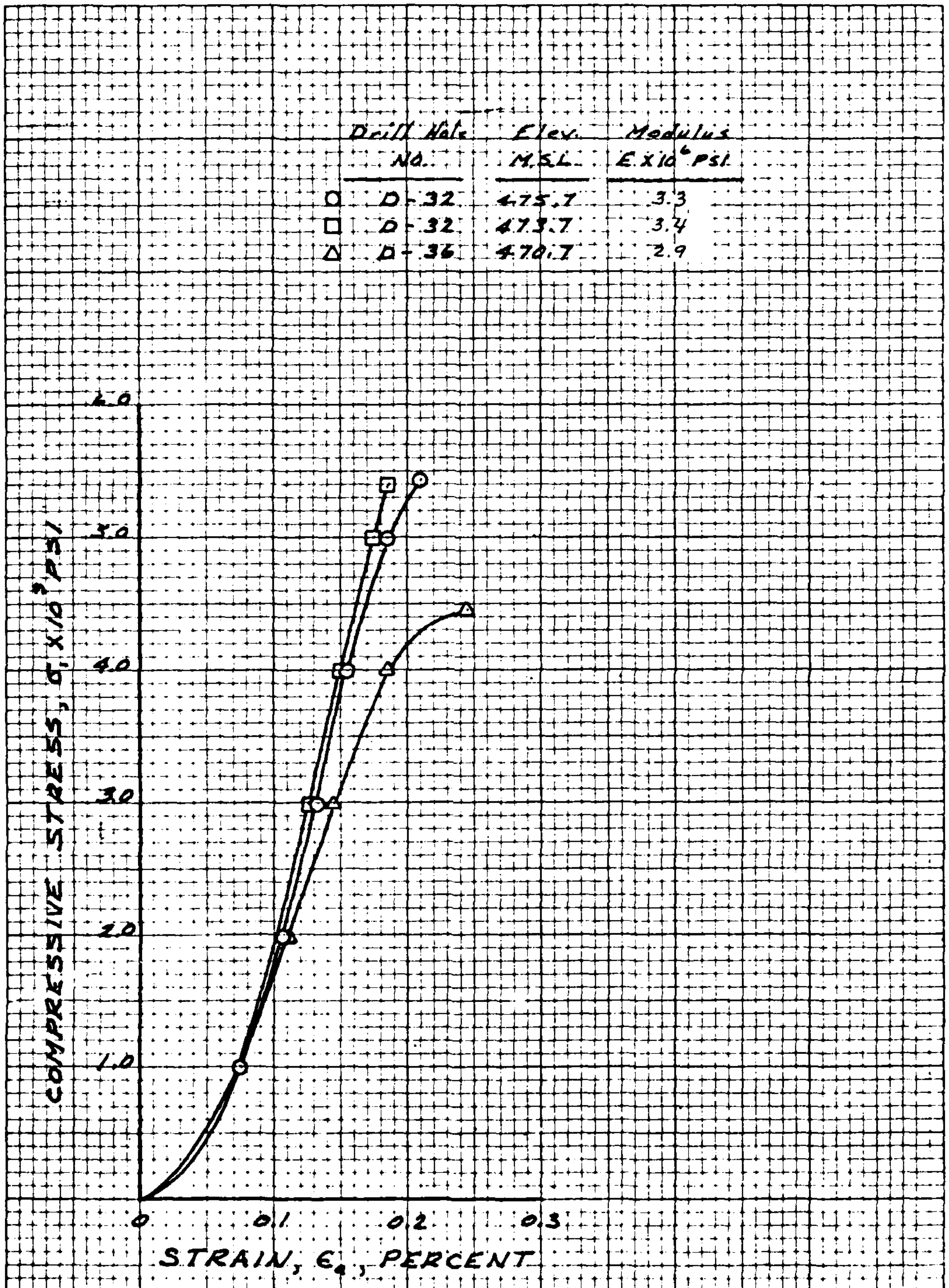
PLATE 20

Hole No.	Elev. FT.	σ_1 PSI	Modulus $E \times 10^6$ psi	Poisson's Ratio	Moisture Percent
D 9	509.1	5830	2.50	0.28	8.6
D 9	491.3	6040	5.00	0.22	6.7
D 9	463.8	5700	4.57	0.21	6.8



STRESS-STRAIN RELATION FOR CONCRETE, DI LOCK AND DAM

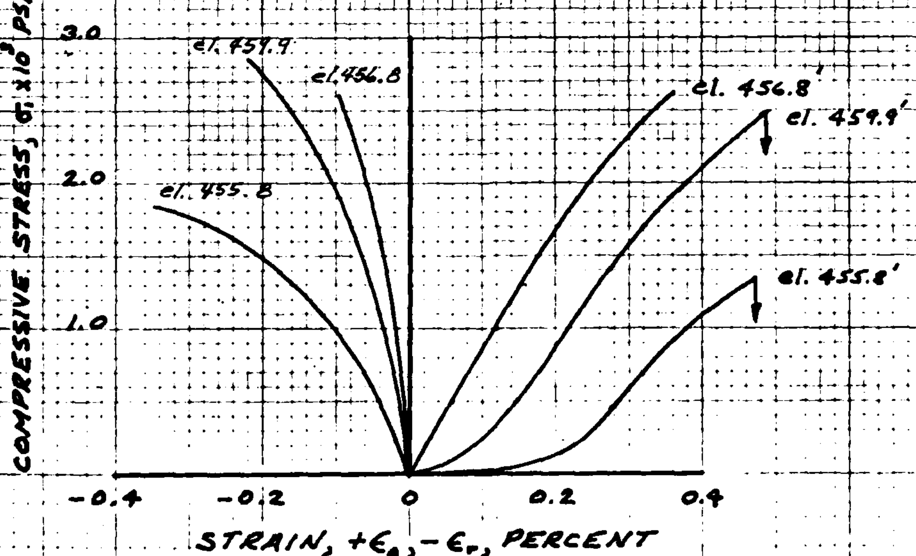
PLATE 21



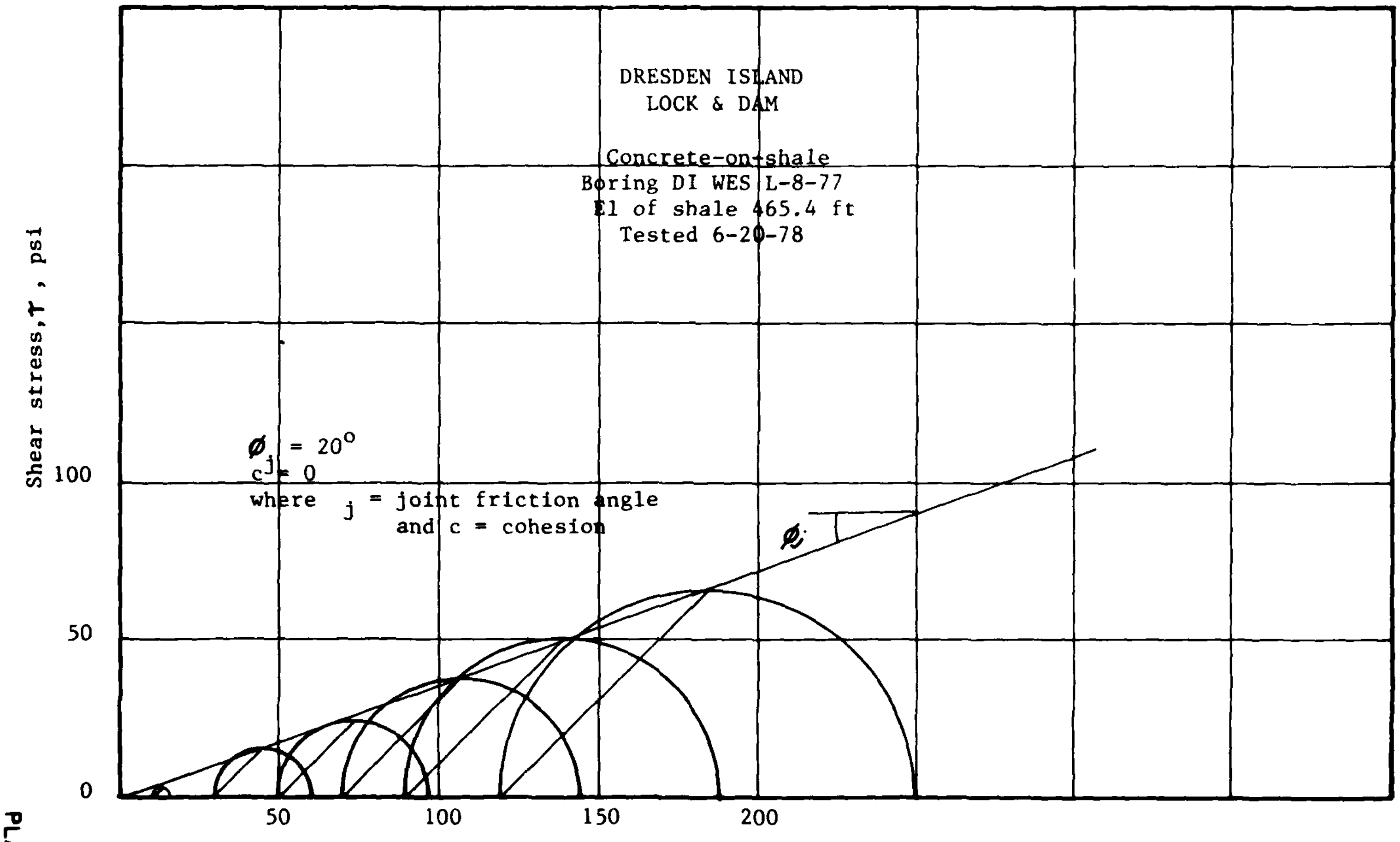
STRESS-STRAIN RELATION FOR VUGGY LIMESTONE, DI LOCK AND DAM

PLATE 22

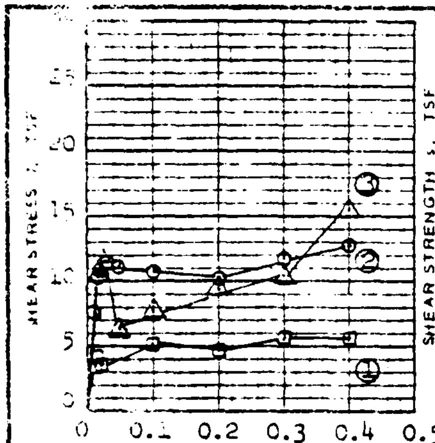
Hole No.	Elev. FT.	σ_1 PSI	Modulus $E \times 10^6$, psi	Poisson's Ratio	Moisture Percent
D9	459.9	2880	0.63	0.17	4.6
D9	456.8	2620	0.86	0.17	5.5
D9	455.8	1830	0.58	0.26	6.1

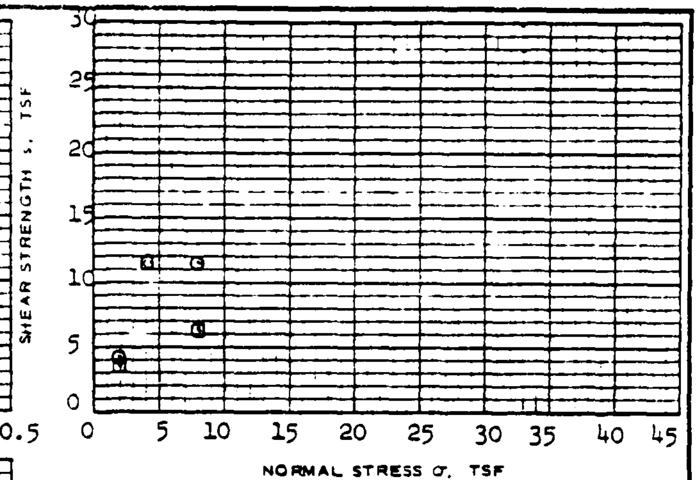


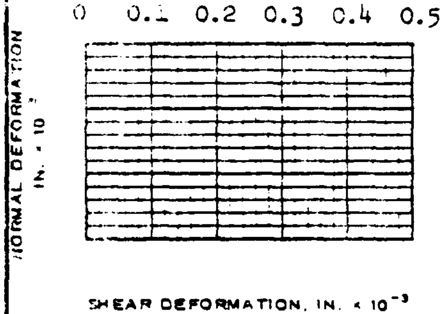
STRESS-STRAIN RELATION FOR SHALE, DI LOCK AND DAM



Mohr diagram for locating stresses on failure plane; multistage triaxial test







SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE	
ϕ =			
TAN ϕ =			
c =			TSF

TEST NO.		1	2	3			
WET DENSITY, PCF	γ_m	153.7	169.4	150.1			
WATER CONTENT	w	6.9 %	5.4 %	8.7 %	%	%	%
TEST SPECIMEN DEPTH		31.6	32.1	37.5			
RATE OF SHEAR INCREASED AT		0.030	0.050	0.050			
NORMAL STRESS, TSF	σ	2.0	4.0	8.0			
MAXIMUM SHEAR STRESS, TSF	τ_f	4.11	11.5	11.4			
TIME TO FAILURE, MINUTES	t_f	8	19	6			
ULTIMATE SHEAR STRESS, TSF	τ_r	3.85	11.1	6.4			
INITIAL DIAMETER, IN.	Do	5.97	5.97	5.98			
INITIAL HEIGHT, IN.	Ho						

DESCRIPTION OF MATERIAL INTACT, GRAY TO BROWN SHALE

REMARKS

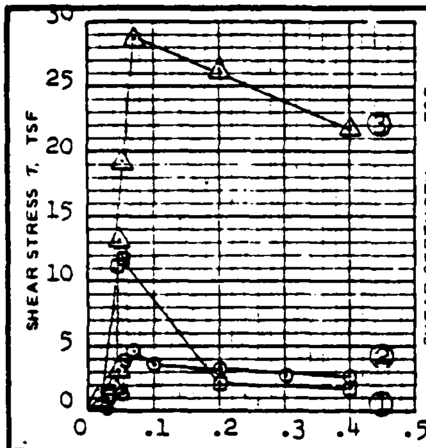
⊡ MAXIMUM SHEAR STRESS

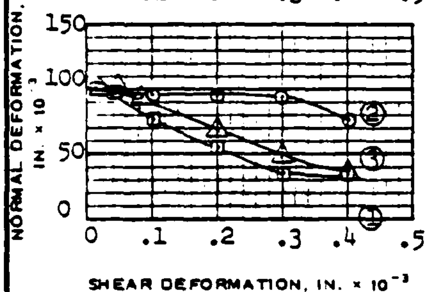
⊙ ULTIMATE SHEAR STRESS

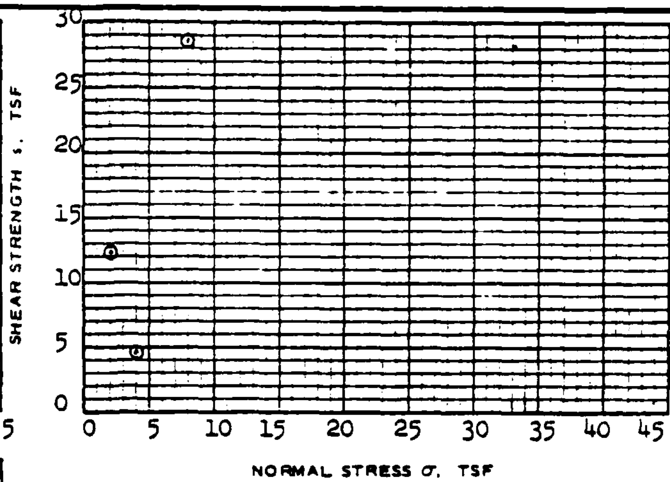
PROJECT DRESDEN ISLAND LOCK AND DAM	
AREA	
BORING NO. GW-2	SAMPLE NO.
DEPTH EL	DATE 20JUL77
TYPE DIRECT SHEAR TEST REPORT (ROCK)	

ENG FORM 4983 PREVIOUS EDITIONS ARE OBSOLETE

PLATE 25







SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE	
ϕ =			
TAN ϕ =			
c =			TSF

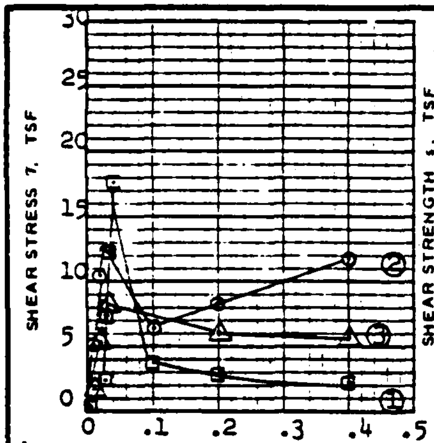
TEST NO.		1	2	3			
WET DENSITY, PCF	γ_m	169.9					
WATER CONTENT	w	4.6 %	%	%	%	%	%
NORMAL STRESS, TSF	σ	2.0	4.0	8.0			
MAXIMUM SHEAR STRESS, TSF	τ_f	12.3	4.0	28.9			
TIME TO FAILURE, MINUTES	t_f	37	13	34			
ULTIMATE SHEAR STRESS, TSF	τ_r	1.9	2.6	21.8			
INITIAL AREA, IN^2		27.2	27.2	27.2			
INITIAL HEIGHT, IN.	Ho	-	-	-			

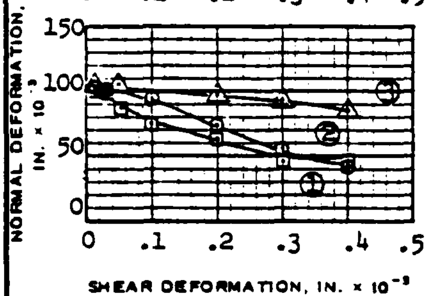
DESCRIPTION OF MATERIAL INTACT, GREEN SHALE (formerly called green clayey shale)

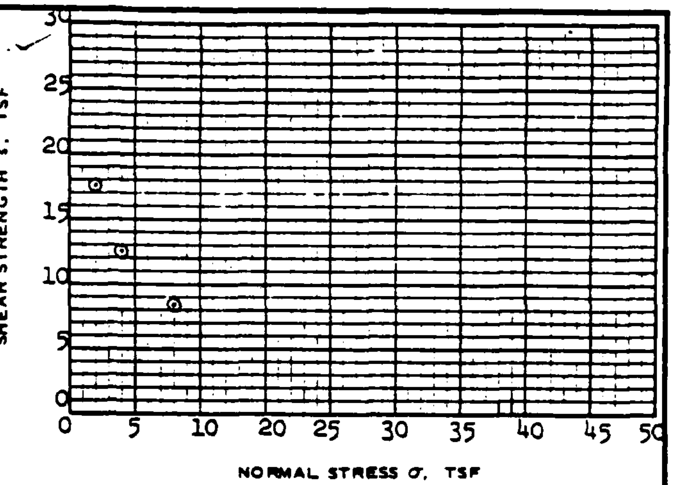
REMARKS Multistage test run on specimen. Maximum shear stress obtained at 4- and 8-tsf normal load not used in plotting peak failure envelope.

PROJECT DRESDEN ISLAND LOCK AND DAM	
AREA	
BORING NO. MCB-5	SAMPLE NO.
DEPTH/EL 21.7 /475.4	DATE 24 JAN 78

DIRECT SHEAR TEST REPORT (ROCK)







SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE	
ϕ =			
TAN ϕ =			
c =			TSF

TEST NO.		1	2	3			
WET DENSITY, PCF	γ_m	168.3					
WATER CONTENT	w	3.8 %	%	%	%	%	%
NORMAL STRESS, TSF	σ	2.0	4.0	8.0			
MAXIMUM SHEAR STRESS, TSF	τ_f	17.8	12.5	8.5			
TIME TO FAILURE, MINUTES	t_f	35	26	8			
ULTIMATE SHEAR STRESS, TSF	τ_r	2.1	11.9	6.0			
INITIAL AREA IN^2		27.8	27.8	27.8			
INITIAL HEIGHT, IN.	Ho	-	-	-			

DESCRIPTION OF MATERIAL INTACT, GREEN SHALE
(formerly called green clayey shale)

REMARKS Multistage test run on specimen. Maximum shear stress obtained at 4- and 8-tsf normal load not used in plotting peak failure envelope.

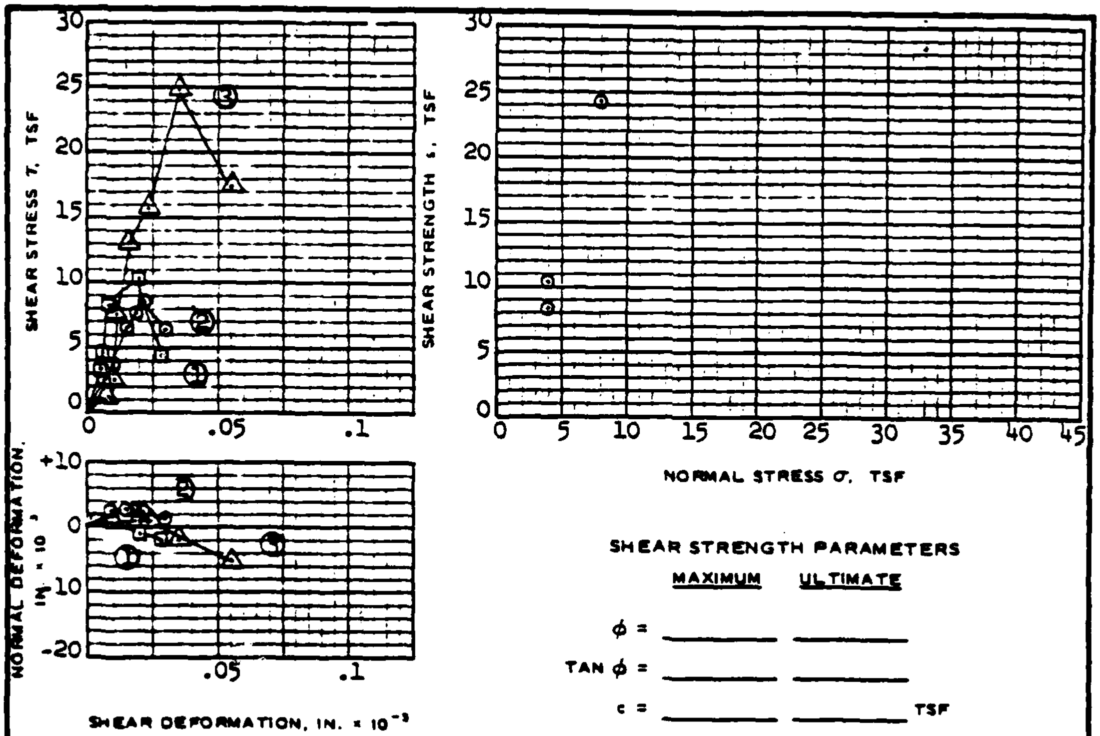
PROJECT DRESDEN ISLAND LOCK AND DAM

AREA

BORING NO. MCB-7 | SAMPLE NO.

DEPTH / EL. 19.1/475.9 | DATE 24 JAN 78

TES DIRECT SHEAR TEST REPORT (ROCK)



SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE
ϕ =		
TAN ϕ =		
c =		TSF

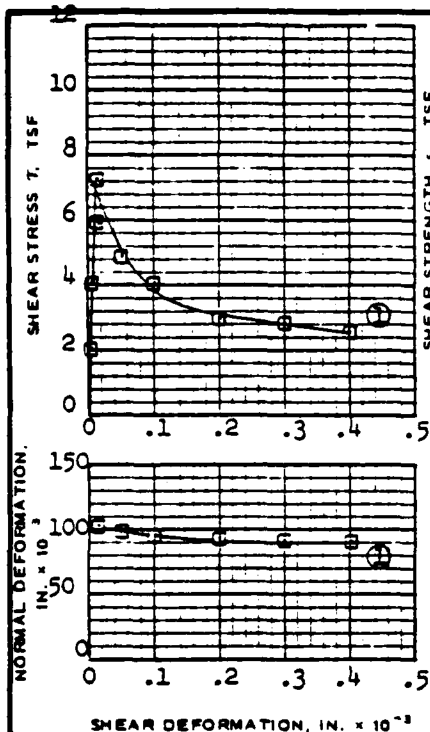
TEST NO.		1	2	3			
WET DENSITY, PCF	γ_m	167.9	157.6	176.4			
WATER CONTENT	w	4.7 %	5.4 %	4.2 %	%	%	%
BORING		D-30-78	D-31-78	D-29-78			
DEPTH		5.8	6.9	4.9			
NORMAL STRESS, TSF	σ	4.0	4.0	8.0			
MAXIMUM SHEAR STRESS, TSF	τ_f	10.2	8.8	24.6			
TIME TO FAILURE, MINUTES	t_f	16	12	26			
ULTIMATE SHEAR STRESS, TSF	τ_r	3.2	3.1	9.9			
INITIAL AREA, IN^2	A_o	27.98	27.55	27.51			
INITIAL HEIGHT, IN.	Ho	-	-	-			

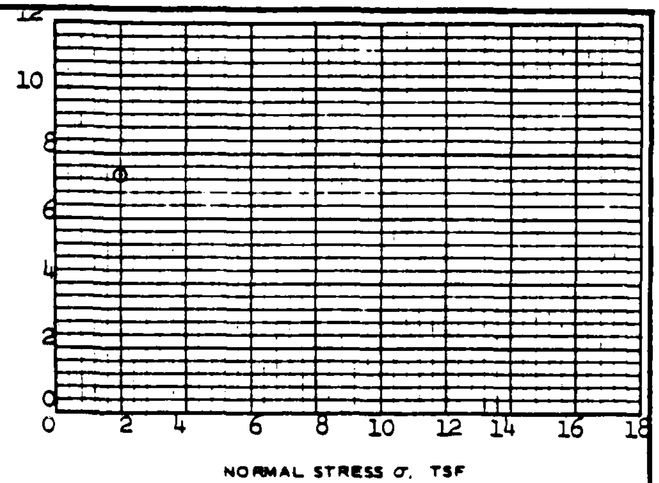
DESCRIPTION OF MATERIAL INTACT, GREEN SHALE
(formerly called green clayey shale)

REMARKS

PROJECT	DRESDEN ISLAND LOCK AND DAM
AREA	
BORING NO. See above	SAMPLE NO.
DEPTH EL. See above	DATE 14 AUG 78
TES	DIRECT SHEAR TEST REPORT (ROCK)

WES FORM APR 75 1490 EDITION OF JUN 65 IS OBSOLETE





SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE
ϕ =		
TAN ϕ =		
c =		TSF

TEST NO.		1					
WET DENSITY, PCF	γ_m	160.5					
WATER CONTENT	w	5.5 %	%	%	%	%	%
NORMAL STRESS, TSF	σ	2.0					
MAXIMUM SHEAR STRESS, TSF	τ_f	7.3					
TIME TO FAILURE, MINUTES	t_f	6					
ULTIMATE SHEAR STRESS, TSF	τ_r	2.6					
INITIAL AREA, IN^2		3.6					
INITIAL HEIGHT, IN.	Ho	-					

DESCRIPTION OF MATERIAL INTACT, GREEN SHALE
(formerly called green clayey shale)

REMARKS

PROJECT DRESDEN ISLAND LOCK AND DAM

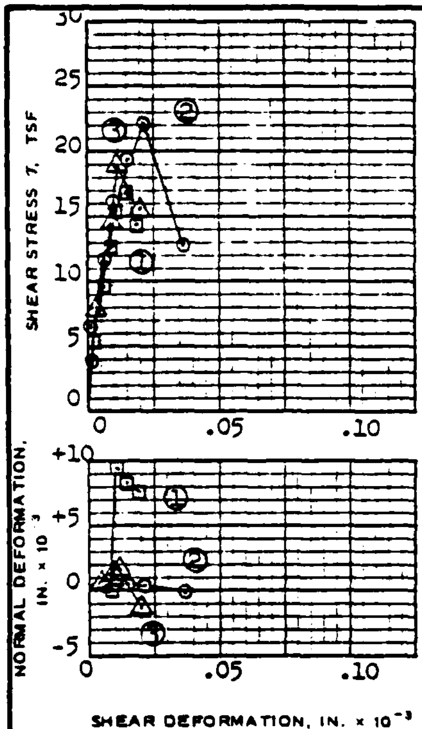
AREA

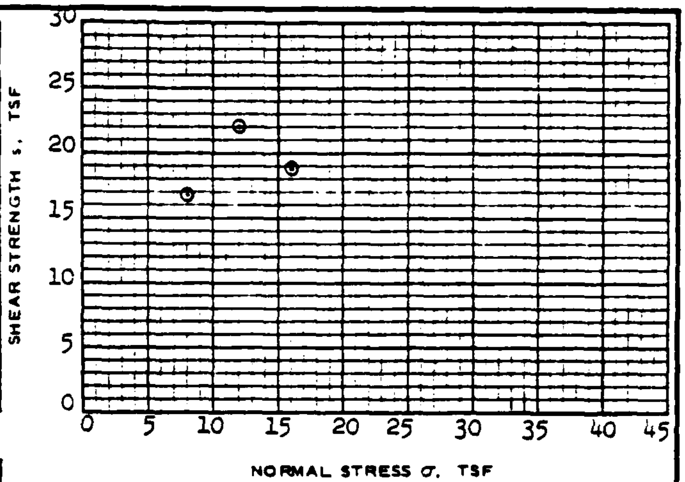
BORING NO. WES-E-2 SAMPLE NO.

DEPTH EL. 50.6 DATE 10 FEB 78

TES DIRECT SHEAR TEST REPORT (ROCK)

WES FORM APR 73 1490 EDITION OF JUN 65 IS OBSOLETE





SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE	
ϕ =			
TAN ϕ =			
c =			TSF

TEST NO.		1	2	3			
WET DENSITY, PCF	γ_m	161.0	160.2	158.8			
WATER CONTENT	w	6.0%	5.0%	5.3%	%	%	%
BORING		D-35-78	D-40-78	D-41-78			
DEPTH		12.5	1.7	6.5			
NORMAL STRESS, TSF	σ	8.0	12.0	16.0			
MAXIMUM SHEAR STRESS, TSF	τ_f	16.9	22.0	18.9			
TIME TO FAILURE, MINUTES	t_f	13	12	21			
ULTIMATE SHEAR STRESS, TSF	τ_r	6.5	11.5	11.1			
INITIAL AREA, IN^2	A_o	27.84	27.79	27.56			
INITIAL HEIGHT, IN.	H_o	-	-	-			

DESCRIPTION OF MATERIAL INTACT, GREEN SHALE
(formerly called green clayey shale)

REMARKS

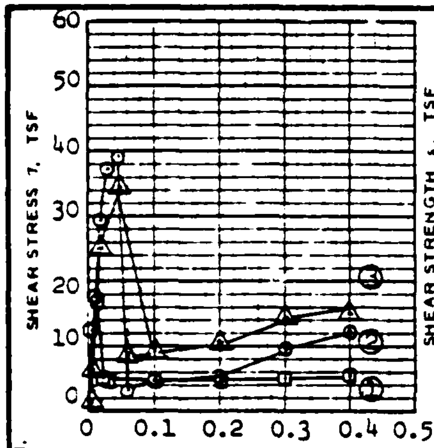
PROJECT DRESDEN ISLAND LOCK AND DAM

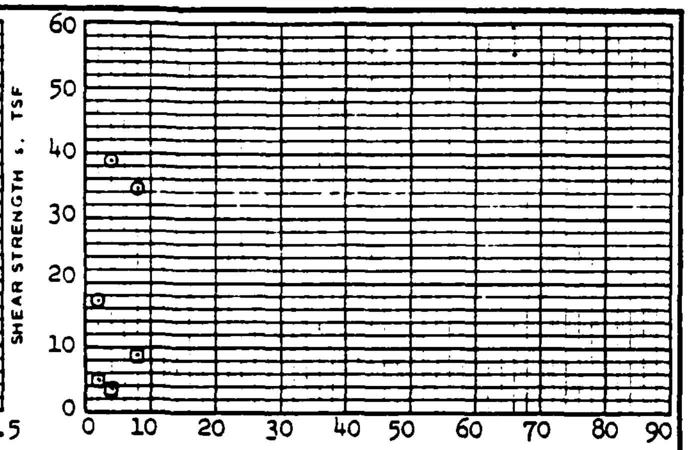
AREA

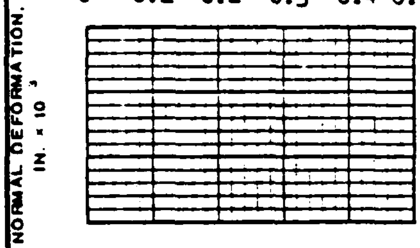
BORING NO. See above — SAMPLE NO.

DEPTH / EL. See above — DATE 14 AUG 78

TES DIRECT SHEAR TEST REPORT (ROCK)







SHEAR DEFORMATION, IN. × 10⁻³

SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE	
ϕ =			
TAN ϕ =			
c =			TSF

TEST NO.		1	2	3			
WET DENSITY, PCF	γ_m	174.0	174.4	174.8			
WATER CONTENT	w	2.2 %	0.9 %	1.9 %	%	%	%
TEST SPECIMEN DEPTH		23.7	25.0	25.2			
RATE OF SHEAR INCREASED AT		0.025	0.070	0.075			
NORMAL STRESS, TSF	σ	2.0	4.0	8.0			
MAXIMUM SHEAR STRESS, TSF	τ_f	16.7	39.2	34.5			
TIME TO FAILURE, MINUTES	t_f	19	23	23			
ULTIMATE SHEAR STRESS, TSF	τ_r	5.1	3.5	9.0			
INITIAL DIAMETER, IN.	Do	5.98	5.96	5.97			
INITIAL HEIGHT, IN.	Ho						

DESCRIPTION OF MATERIAL: CONCRETE ON ROCK, LIMESTONE

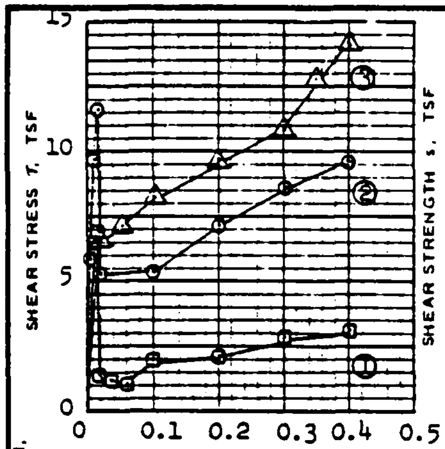
REMARKS

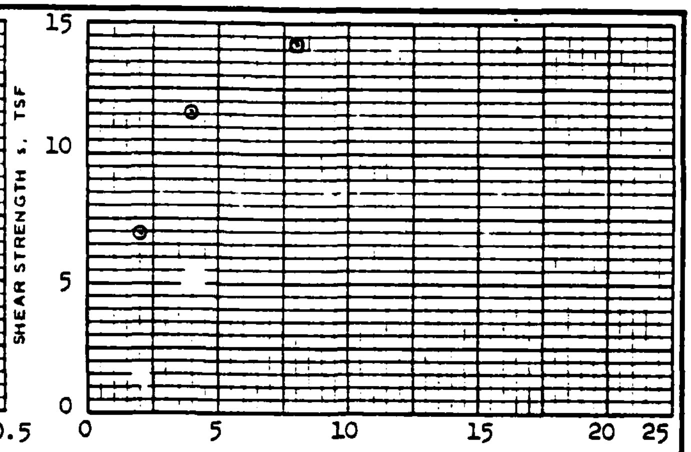
○ MAXIMUM SHEAR STRESS

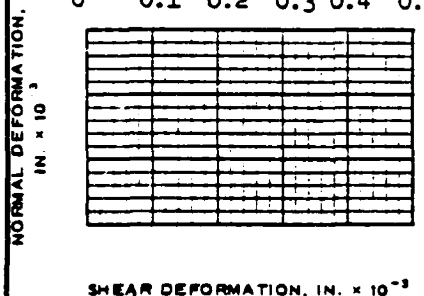
□ ULTIMATE SHEAR STRESS

PROJECT	DRESDEN ISLAND LOCK AND DAM		
AREA			
BORING NO.	GW-2	SAMPLE NO.	
DEPTH EL		DATE	25JUL77
RCH	DIRECT SHEAR TEST REPORT (ROCK)		

WES FORM APR 75 1490 EDITION OF JUN 65 IS OBSOLETE







SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE	
ϕ =	49°		
TAN ϕ =	1.1600		
c =	5.6		TSF

TEST NO.		1	2	3			
WET DENSITY, PCF	γ_m	152.9	156.3	150.3			
WATER CONTENT	w	8.1 %	3.9 %	6.2 %	%	%	%
TEST SPECIMEN DEPTH		26.0	27.4	30.7			
RATE OF SHEAR INCREASED AT		0.040	0.033	NO INCR.			
NORMAL STRESS, TSF	σ	2.0	4.0	8.0			
MAXIMUM SHEAR STRESS, TSF	τ_f	6.9	11.6	14.3			
TIME TO FAILURE, MINUTES	t_f	19	6	43			
ULTIMATE SHEAR STRESS, TSF	τ_r	1.40	5.43	14.3			
INITIAL DIAMETER, IN.	Do	5.93	5.94	5.83			
INITIAL HEIGHT, IN.	Ho						

DESCRIPTION OF MATERIAL CONCRETE ON ROCK, GRAY TO BROWN SHALE

REMARKS
O MAXIMUM SHEAR STRESS

PROJECT DRESDEN ISLAND
LOCK AND DAM

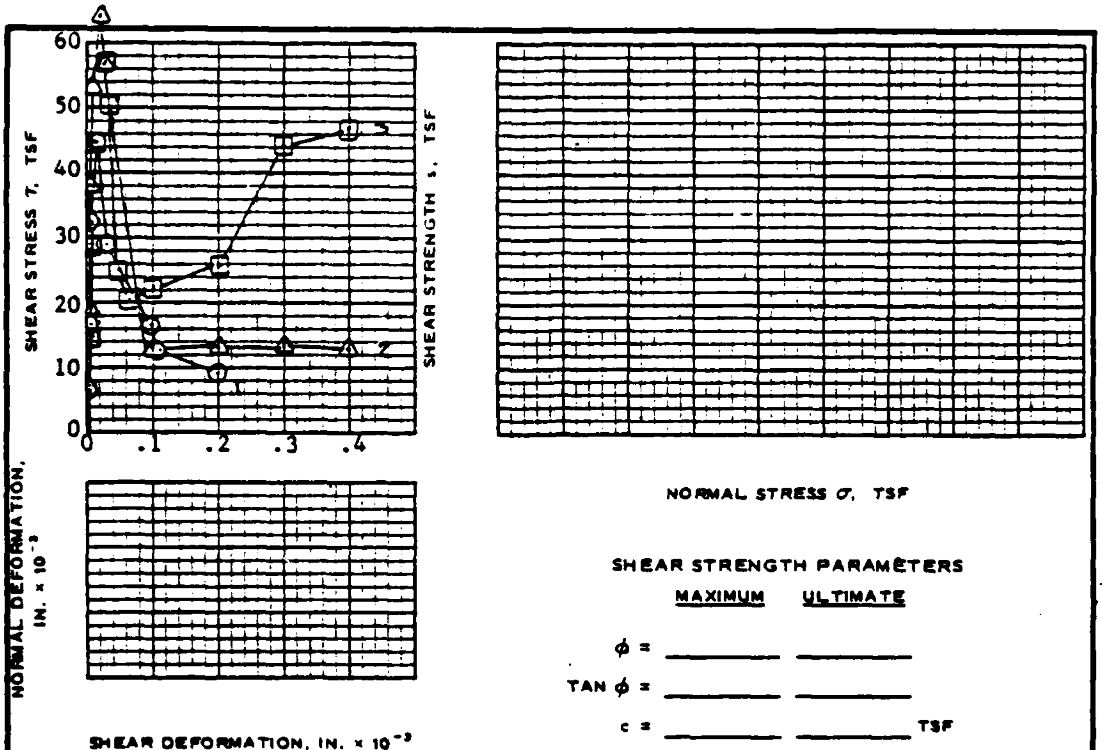
AREA

BORING NO. GW-5 SAMPLE NO.

DEPTH
EL

DATE 29JUL77

RCH DIRECT SHEAR TEST REPORT (ROCK)



SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE	
ϕ =			
TAN ϕ =			
c =			TSF

TEST NO. BORING NO. / ELEVATION, ft		D-38 472.0	D-38 473.1	D-38 473.4			
WET DENSITY, PCF	γ_d	--	--	--			
WATER CONTENT	w	-- %	-- %	-- %	%	%	%
NORMAL STRESS, TSF	σ	2.0	4.0	8.0			
MAXIMUM SHEAR STRESS, TSF	τ_f	45.7	66.0	57.6			
TIME TO FAILURE, MINUTES	t_f	5	8	10			
ULTIMATE SHEAR STRESS, TSF	τ_r	9.8	11.5	20.7			
INITIAL DIAMETER, IN.	Do	11.04	9.39	12.19			
INITIAL HEIGHT, IN.	Ho	--	--	--			

DESCRIPTION OF MATERIAL CROSS-BED, LIMESTONE

REMARKS

PROJECT DRESDEN ISLAND LOCK & DAM
COMPLIANCE & SCOUR DETECTION

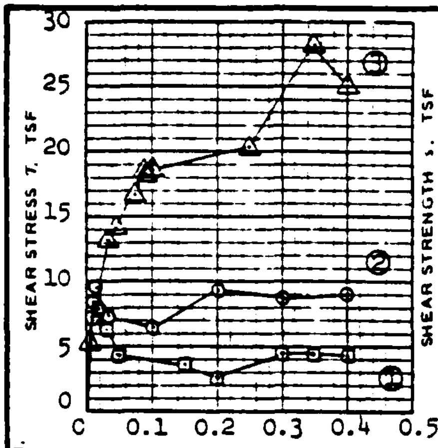
AREA

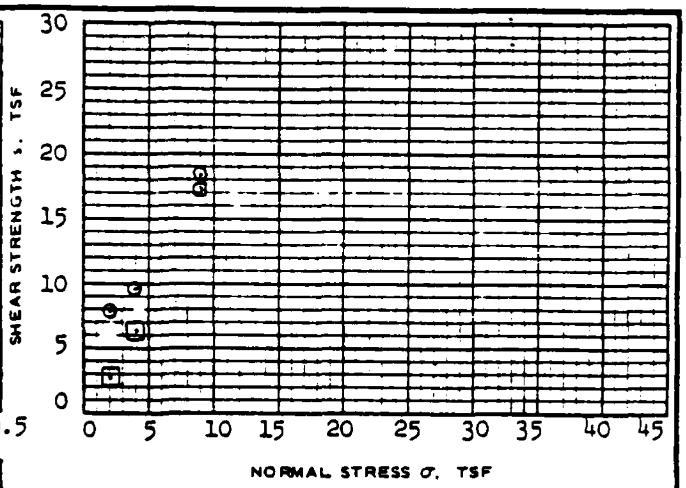
BORING NO. See Test No. SAMPLE NO.

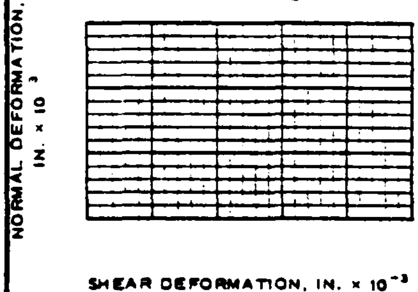
DEPTH EL See Test No. DATE 15 May 79

DIRECT SHEAR TEST REPORT (ROCK)

WES FORM APR 76 1490 EDITION OF JUN 65 IS OBSOLETE







SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE	
ϕ =			
TAN ϕ =			
c =			TSF

TEST NO.		1	2	3			
WET DENSITY, PCF	γ_m	151.6	165.2	158.1			
WATER CONTENT	w	8.5 %	4.0 %	5.8 %	%	%	%
TEST SPECIMEN DEPTH		35.4	40.5	44.0			
RATE OF SHEAR INCREASED AT		0.025	0.030	0.085			
NORMAL STRESS, TSF	σ	2.0	4.0	8.0			
MAXIMUM SHEAR STRESS, TSF	τ_f	7.9	9.6	18.5			
TIME TO FAILURE, MINUTES	t_f	6	8	9			
ULTIMATE SHEAR STRESS, TSF	τ_r	2.8	6.5	17.6			
INITIAL DIAMETER, IN.	Do	14.34	11.44	10.36			
INITIAL HEIGHT, IN.	Ho						

DESCRIPTION OF MATERIAL CROSS-BEDDED, GRAY TO BROWN SHALE

REMARKS
○ MAXIMUM SHEAR STRESS
□ ULTIMATE SHEAR STRESS

PROJECT DRESDEN ISLAND LOCK AND DAM

AREA

BORING NO. GW-2

SAMPLE NO.

DEPTH EL.

DATE 03AUG77

RCH DIRECT SHEAR TEST REPORT (ROCK)

WES FORM APR 73 1490 EDITION OF JUN 65 IS OBSOLETE

SHEAR STRESS τ, TSF

SHEAR STRENGTH s, TSF

NORMAL DEFORMATION, IN. × 10^{-3}

SHEAR DEFORMATION, IN. × 10^{-3}

NORMAL STRESS σ, TSF

SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE	
ϕ =			
TAN ϕ =			
c =			TSF

TEST NO. BORING NO. / ELEVATION, FT		RB-4 479.6	RB-2 475.4	RB-2 478.7	RB-5 481.5		
WET DENSITY, PCF	γ_d	170.1	172.3	168.8	157.7		
WATER CONTENT	w	2.4%	4.1 %	2.8%	1.8 %	%	%
NORMAL STRESS, TSF	σ	1.5	2.5	4.0	8.0		
MAXIMUM SHEAR STRESS, TSF	τ_f	3.41	2.01	5.5	—		
TIME TO FAILURE, MINUTES	t_f						
ULTIMATE SHEAR STRESS, TSF	τ_r	2.0	1.1	1.7	4.3		
INITIAL DIAMETER, IN.	Do						
INITIAL HEIGHT, IN.	Ho						

DESCRIPTION OF MATERIAL FILLED PARTING, CLAY SEAM IN LIMESTONE

REMARKS

PROJECT DRESDEN ISLAND LOCK & DAM
COMPLIANCE & SCOUR DETECTION

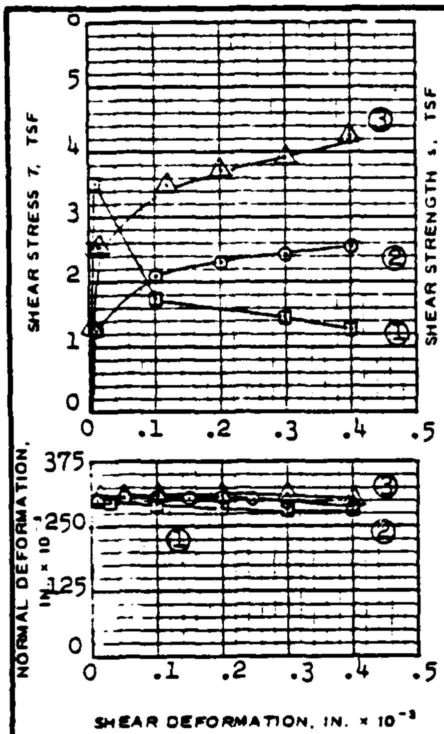
AREA

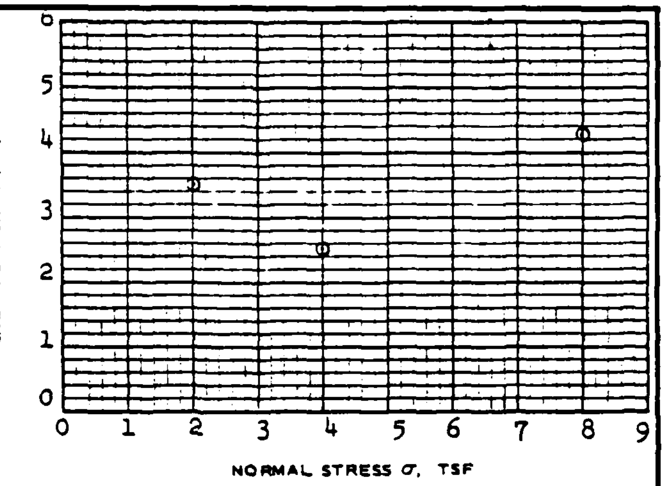
BORING NO. See Test No. SAMPLE NO.

DEPTH EL. See Test No. DATE Aug 1980

DIRECT SHEAR TEST REPORT (ROCK)

WES FORM APR 75 1490 EDITION OF JUN 65 IS OBSOLETE





SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE	
φ =			
TAN φ =			
c =			TSF

TEST NO.		1	2	3			
WET DENSITY, PCF	γ_m	161.2					
WATER CONTENT	w	3.9 %	%	%	%	%	%
NORMAL STRESS, TSF	σ	2.0	4.0	8.0			
MAXIMUM SHEAR STRESS, TSF	τ_f	3.5	2.6	4.3			
TIME TO FAILURE, MINUTES	t_f	19	71	64			
ULTIMATE SHEAR STRESS, TSF	τ_r	1.3	2.6	4.3			
INITIAL AREA IN^2		27.79	27.79	27.79			
INITIAL HEIGHT, IN.	H_o	-	-	-			

DESCRIPTION OF MATERIAL MULTI-STAGE SHEAR ON CLAY SEAM IN LIMESTONE
ATTERBERG LIMITS ON MATERIAL AT 28.6': LL=34, PL=14, PI=20

REMARKS

PROJECT DRESDEN ISLAND LOCK AND DAM

AREA

BORING NO. GWB-2 SAMPLE NO.

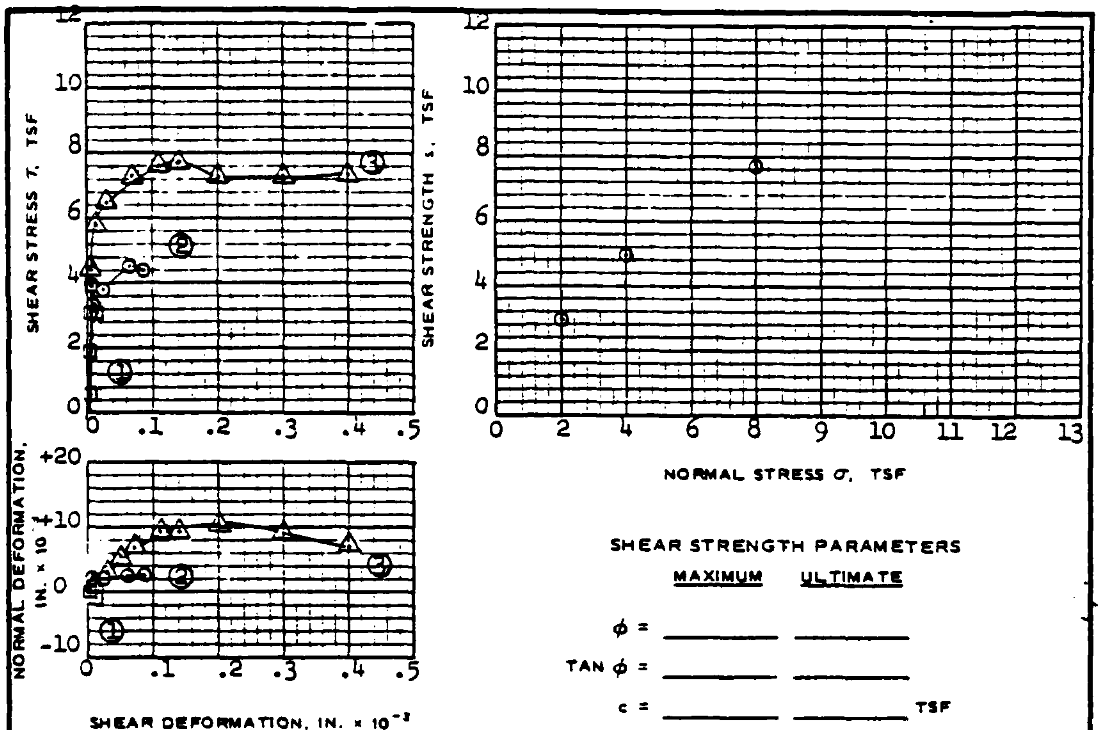
DEPTH EL 28.95 /481.3 DATE 13 FEB 78

TES DIRECT SHEAR TEST REPORT (ROCK)

WES FORM APR 75 1490 EDITION OF JUN 65 IS OBSOLETE

PLATE 36a

SHEET NO.



SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE
φ =		
TAN φ =		
c =		TSF

TEST NO.		1	2	3			
WET DENSITY, PCF	γ_m	159.1					
WATER CONTENT	w	15.1%	%	%	%	%	%
NORMAL STRESS, TSF	σ	2.0	4.0	8.0			
MAXIMUM SHEAR STRESS, TSF	τ_f	2.98	4.47	7.73			
TIME TO FAILURE, MINUTES	t_f	5	19	30			
ULTIMATE SHEAR STRESS, TSF	τ_r	2.59	4.40	7.37			
INITIAL AREA, IN^2	A_o	27.79	27.79	27.79			
INITIAL HEIGHT, IN.	Ho	-	-	-			

DESCRIPTION OF MATERIAL LEAN CLAY(CL), GREEN (1/2 IN. GREEN CLAY SEAM IN LIMESTONE)

REMARKS

PROJECT DRESDEN ISLAND LOCK AND DAM

AREA

BORING NO. DI-WES SAMPLE NO. D-35-78

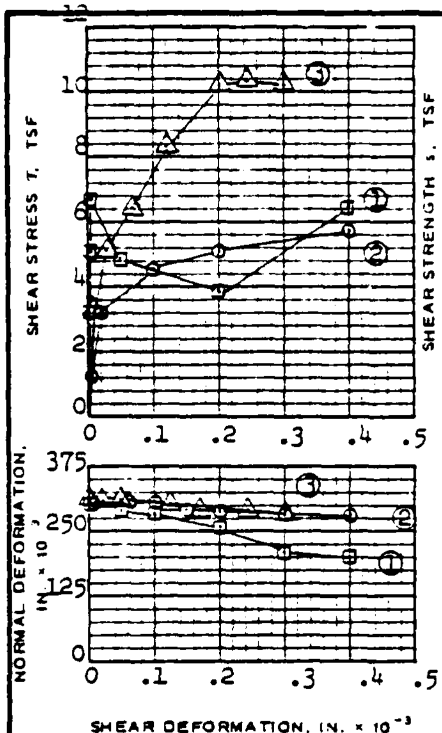
DEPTH EL 8.25 DATE 14 AUG 78

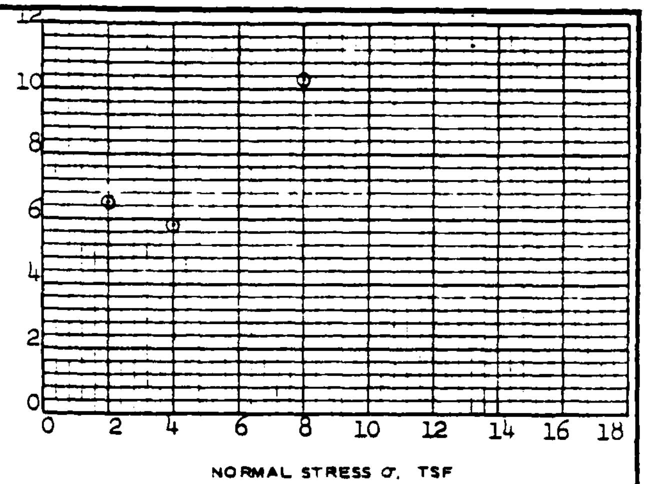
TES DIRECT SHEAR TEST REPORT (ROCK)

WES FORM APR 73 1490 EDITION OF JUN 65 IS OBSOLETE

PLATE 36b

SHEET NO.





SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE	
ϕ =	35.5°		
TAN ϕ =			
c =	4.2		TSF

TEST NO.		1	2	3			
WET DENSITY, PCF	γ_m	156.0					
WATER CONTENT	w	2.9 %	%	%	%	%	%
NORMAL STRESS, TSF	σ	2.0	4.0	8.0			
MAXIMUM SHEAR STRESS, TSF	τ_f	6.6	5.6	10.4			
TIME TO FAILURE, MINUTES	t_f	21	31	46			
ULTIMATE SHEAR STRESS, TSF	τ_r	6.4	5.6	10.2			
INITIAL AREA, IN^2		27.9	27.9	27.9			
INITIAL HEIGHT, IN.	Ho	-	-	-			

DESCRIPTION OF MATERIAL MULTI-STAGE SHEAR ON CLAY SEAM IN BROWN LIMESTONE

REMARKS

PROJECT DRESDEN ISLAND LOCK AND DAM

AREA

BORING NO. GWB-2 SAMPLE NO.

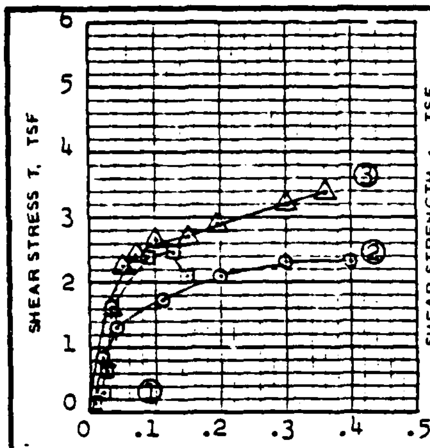
DEPTH / EL 23.25 /487.0 DATE 14 FEB 78

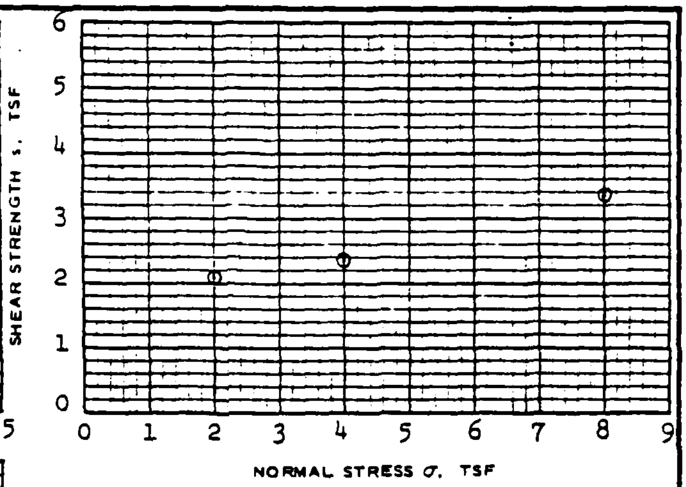
TES DIRECT SHEAR TEST REPORT (ROCK)

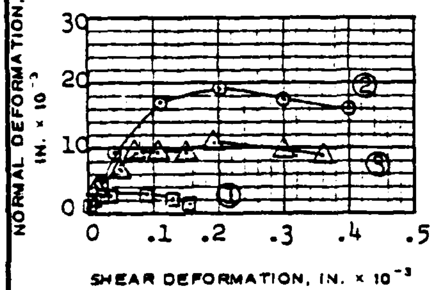
WES FORM APR 73 1490 EDITION OF JUN 65 IS OBSOLETE

PLATE 37a

SHEET NO.







SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE	
ϕ =			
TAN ϕ =			
c =			TSF

TEST NO.		1	2	3			
WET DENSITY, PCF	γ_m	144.2					
WATER CONTENT	w	10.4 %	%	%	%	%	%
NORMAL STRESS, TSF	σ	2.0	4.0	8.0			
MAXIMUM SHEAR STRESS, TSF	τ_f	2.47	2.34	3.36			
TIME TO FAILURE, MINUTES	t_f	31	37	47			
ULTIMATE SHEAR STRESS, TSF	τ_r	2.08	2.34	3.36			
INITIAL AREA, IN^2	A_o	22.47	22.47	22.47			
INITIAL HEIGHT, IN.	Ho	-	-	-			

DESCRIPTION OF MATERIAL CLAY (CL), GREEN, MULTI-STAGE

REMARKS

PROJECT DRESDEN ISLAND LOCK AND DAM

AREA

BORING NO. DI-WES

SAMPLE NO. D-29-78

DEPTH/EL. 3.9/469.1

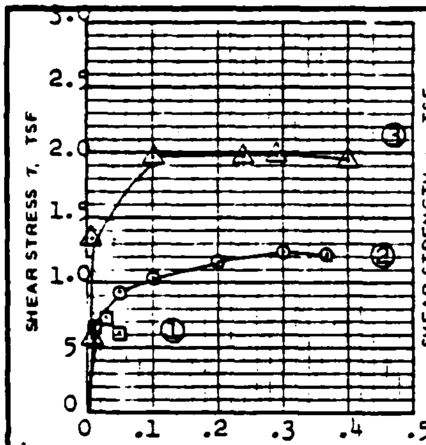
DATE 14 AUG 78

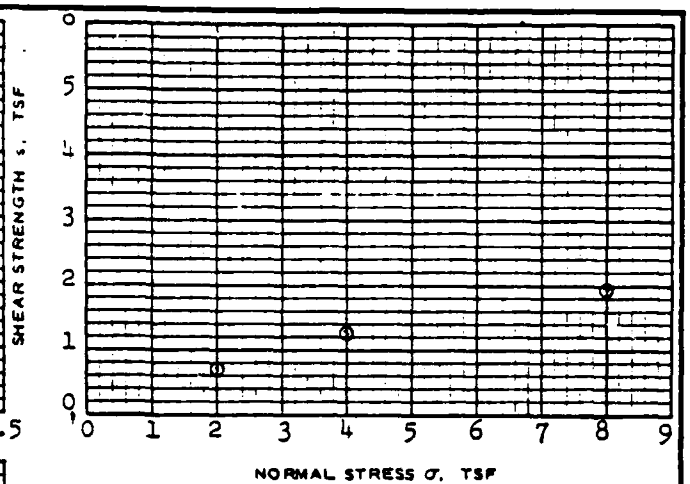
WES DIRECT SHEAR TEST REPORT (ROCK)

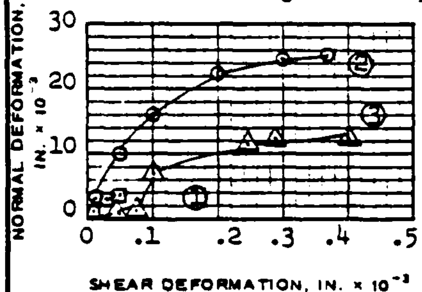
WES FORM APR 73 1490 EDITION OF JUN 65 IS OBSOLETE

PLATE 37b

SHEET NO.







SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE	
ϕ =	11.5°		
TAN ϕ =			
c =	0.4		TSF

TEST NO.		1	2	3			
WET DENSITY, PCF	γ_m	133.0					
WATER CONTENT	w	12.8 %	%	%	%	%	%
NORMAL STRESS, TSF	σ	2.0	4.0	8.0			
MAXIMUM SHEAR STRESS, TSF	τ_f	0.737	1.24	1.98			
TIME TO FAILURE, MINUTES	t_f	4	14	27			
ULTIMATE SHEAR STRESS, TSF	τ_r	0.643	1.21	1.94			
INITIAL AREA, IN^2	Ao	26.86	26.86	26.86			
INITIAL HEIGHT, IN.	Ho	-	-	-			

DESCRIPTION OF MATERIAL CLAY (CL), GREEN, MULTI-STAGE

REMARKS

PROJECT DRESDEN ISLAND LOCK AND DAM

AREA

BORING NO. DI-WES SAMPLE NO. D-35-78

DEPTH EL. 11.2/462.8 DATE 14 AUG 78

TES DIRECT SHEAR TEST REPORT (ROCK)

WES FORM APR 73 1490 EDITION OF JUN 65 IS OBSOLETE

PLATE 37c

SHEET NO.

SHEAR STRESS τ, TSF

SHEAR STRENGTH s, TSF

NORMAL DEFORMATION, IN. × 10^{-3}

SHEAR DEFORMATION, IN. × 10^{-3}

NORMAL STRESS σ, TSF

SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE	
φ =			
TAN φ =			
c =			TSF

TEST NO.							
WET DENSITY, PCF	γ_d	133.0					
WATER CONTENT	w	10.2%	%	%	%	%	%
NORMAL STRESS, TSF	σ	1.5	2.5	4.0			
MAXIMUM SHEAR STRESS, TSF	τ_f						
TIME TO FAILURE, MINUTES	t_f						
ULTIMATE SHEAR STRESS, TSF	τ_r	0.66	1.20	1.77			
INITIAL DIAMETER, IN.	Do						
INITIAL HEIGHT, IN.	Ho						

DESCRIPTION OF MATERIAL FILLED PARTING, COMPOSITE SAMPLE FROM THE CLAY SEAMS IN LIMESTONE PREVIOUSLY TESTED

REMARKS Composite from test specimens:

RB-4 479.6
RB-2 475.4
RB-2 478.7
RB-5 481.5

PROJECT DRESDEN ISLAND LOCK & DAM
COMPLIANCE & SCOUR DETECTION

AREA

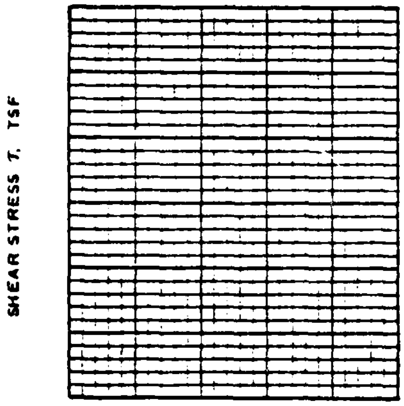
BORING NO. | SAMPLE NO.

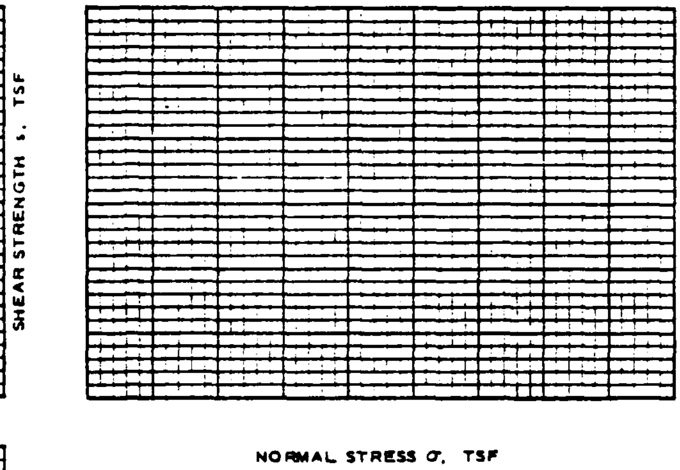
DEPTH EL. | DATE

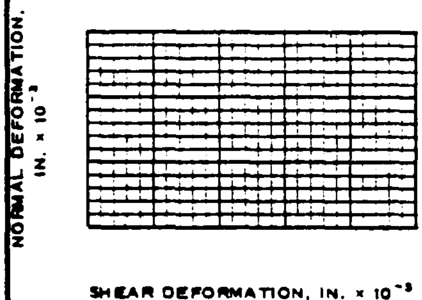
DIRECT SHEAR TEST REPORT (ROCK)

WES FORM APR 73 1490 EDITION OF JUN 65 IS OBSOLETE

SHEET NO.







SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE	
ϕ =			
TAN ϕ =			
c =			TSF

TEST NO. BORING NO. / ELEVATION, ft		RB-2-79 480.7	RB-2-79 480.3	RB-2-79 477.9	RB-4-79 483.1		
WET DENSITY, PCF	γ_d	171.5	165.8	168.2	168.8		
WATER CONTENT	w	3.1 %	1.4%	2.2 %	1.6%	%	%
NORMAL STRESS, TSF	σ	1.50	2.50	4.00	8.00		
MAXIMUM SHEAR STRESS, TSF	τ_f	2.67	—	5.78	7.06		
TIME TO FAILURE, MINUTES	t_f						
ULTIMATE SHEAR STRESS, TSF	τ_r	0.67	1.30	1.60	5.00		
INITIAL DIAMETER, IN.	Do						
INITIAL HEIGHT, IN.	Ho						

DESCRIPTION OF MATERIAL FILLED PARTING, SHALE/CLAY IN LIMESTONE

REMARKS

PROJECT DRESDEN ISLAND LOCK AND DAM
COMPLIANCE AND SCOUR DETECTION

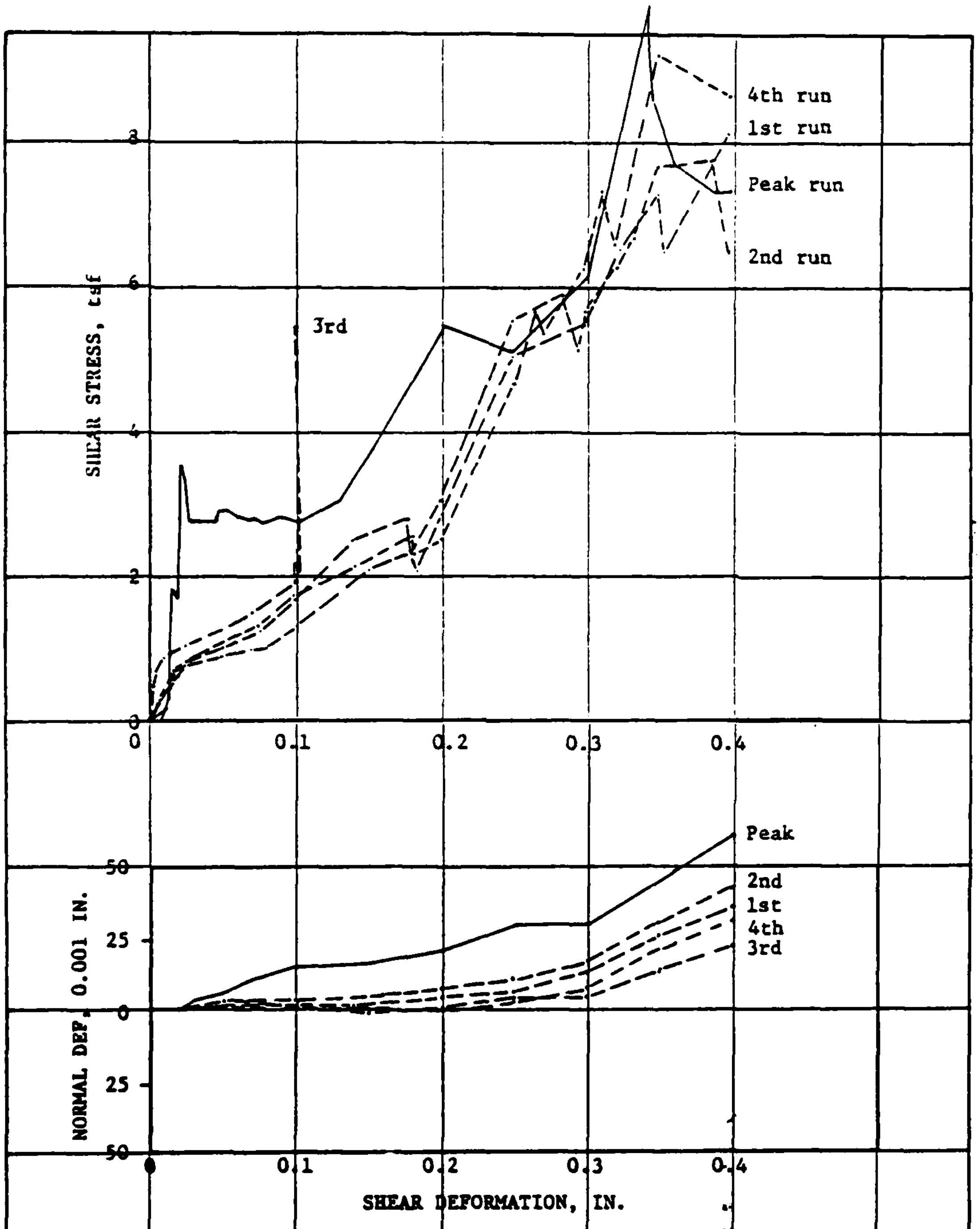
AREA

BORING NO. See Test No. SAMPLE NO.

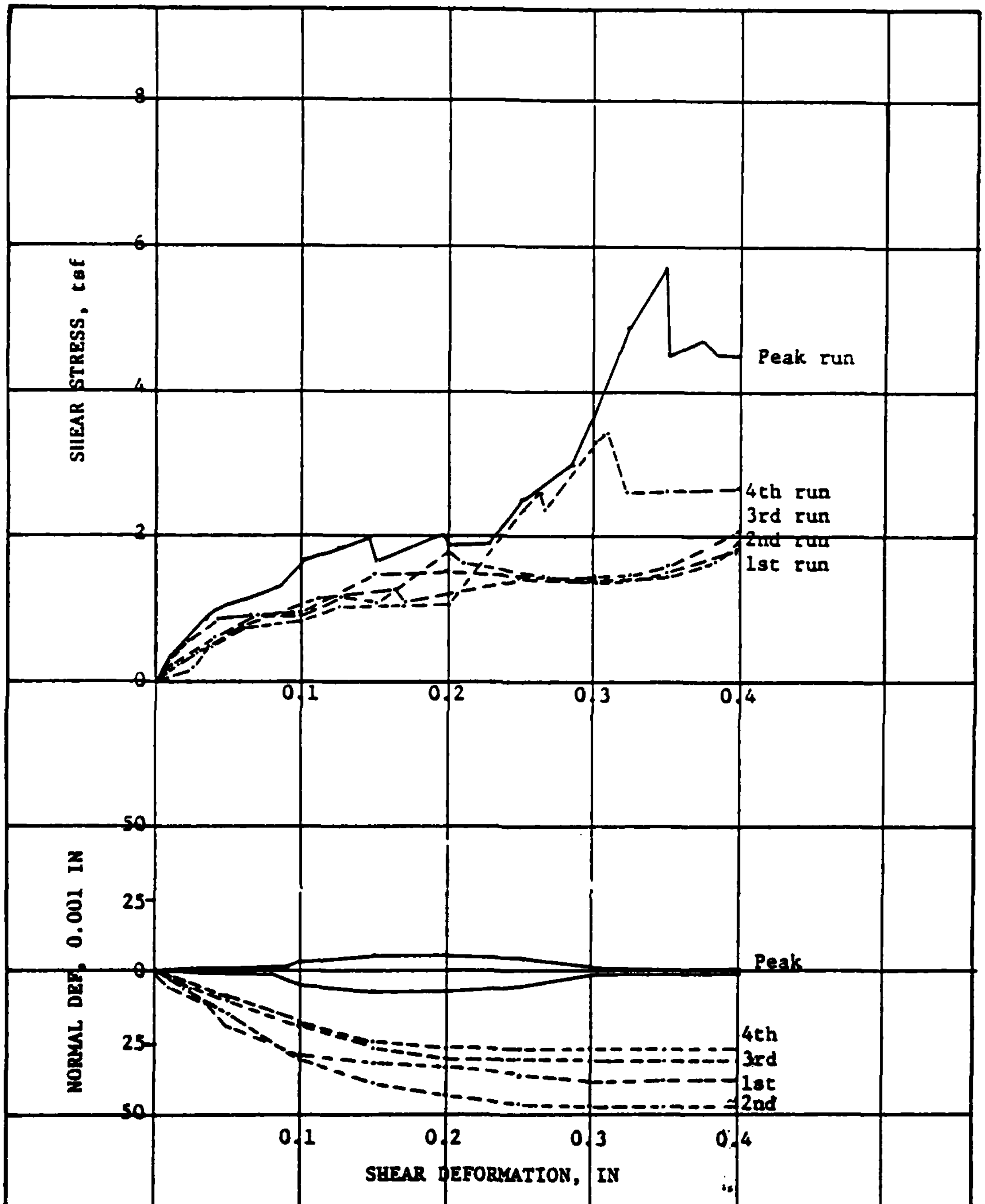
DEPTH / EL. See Test No. DATE Aug 1980

DIRECT SHEAR TEST REPORT (ROCK)

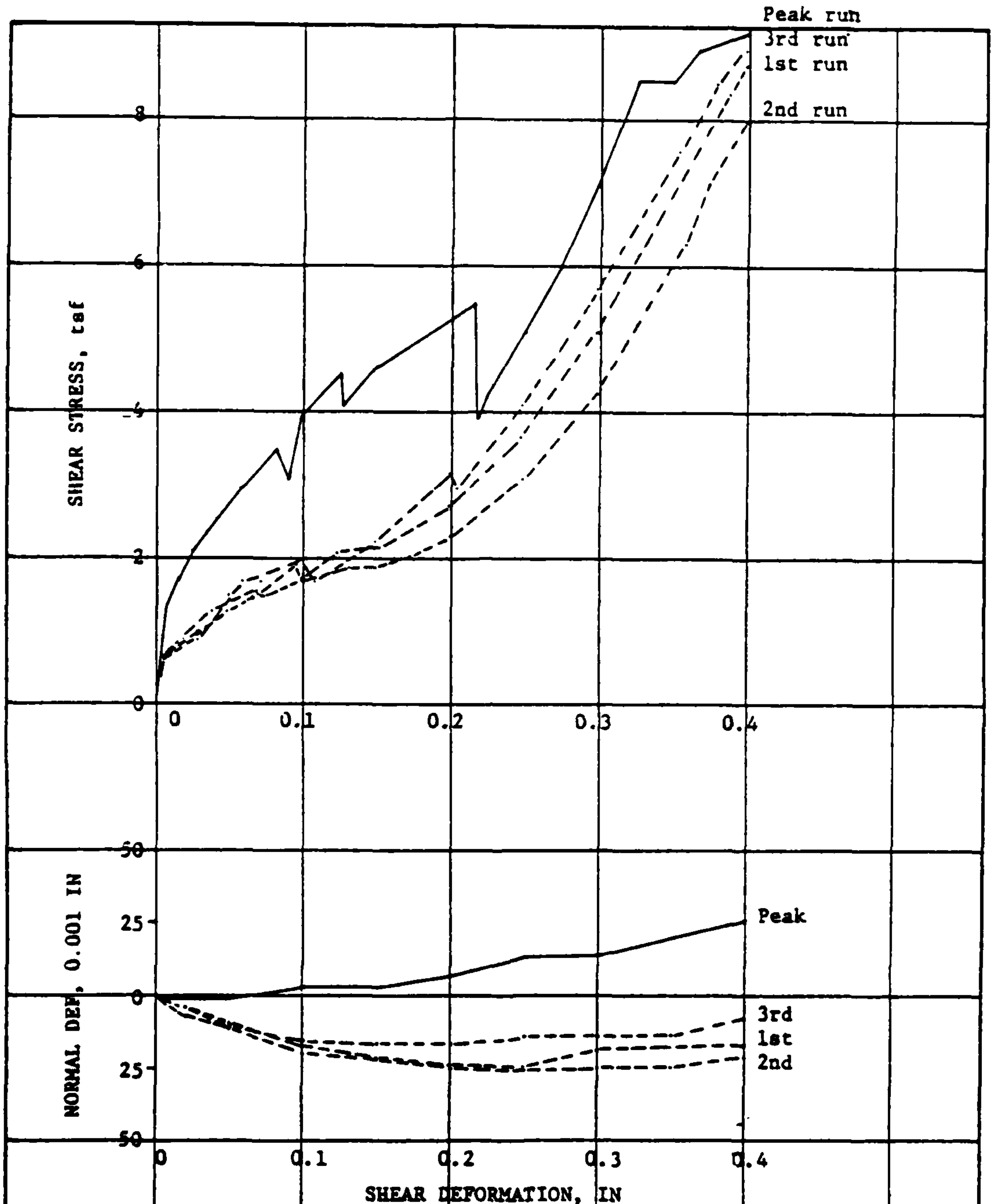
WES FORM APR 73 1490 EDITION OF JUN 65 IS OBSOLETE



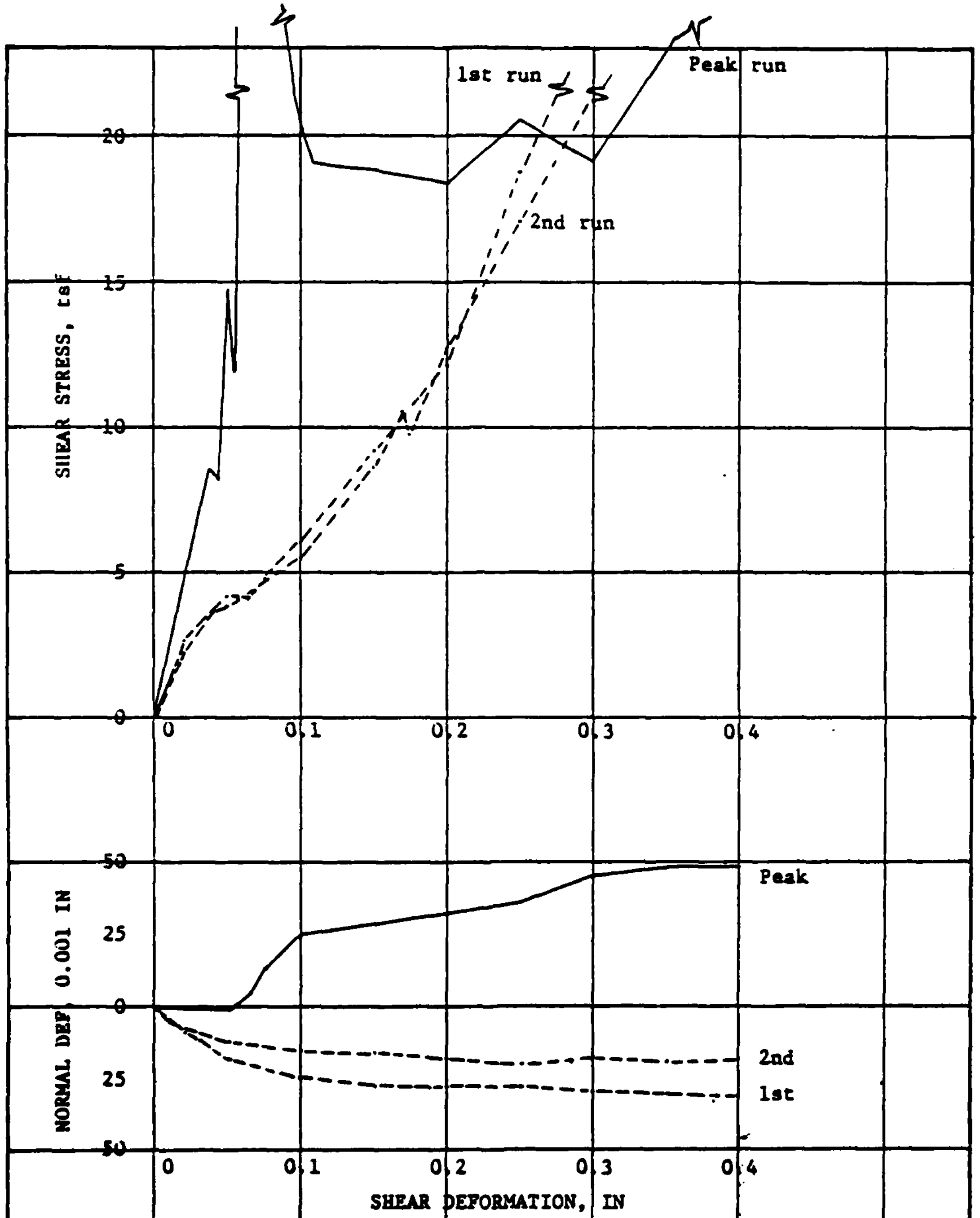
DIRECT SHEAR - CLAY FILLED PARTING IN LIMESTONE
BORINGS DIWES RB-4-79, EL 479.6', σ_n = 1.5 tsf



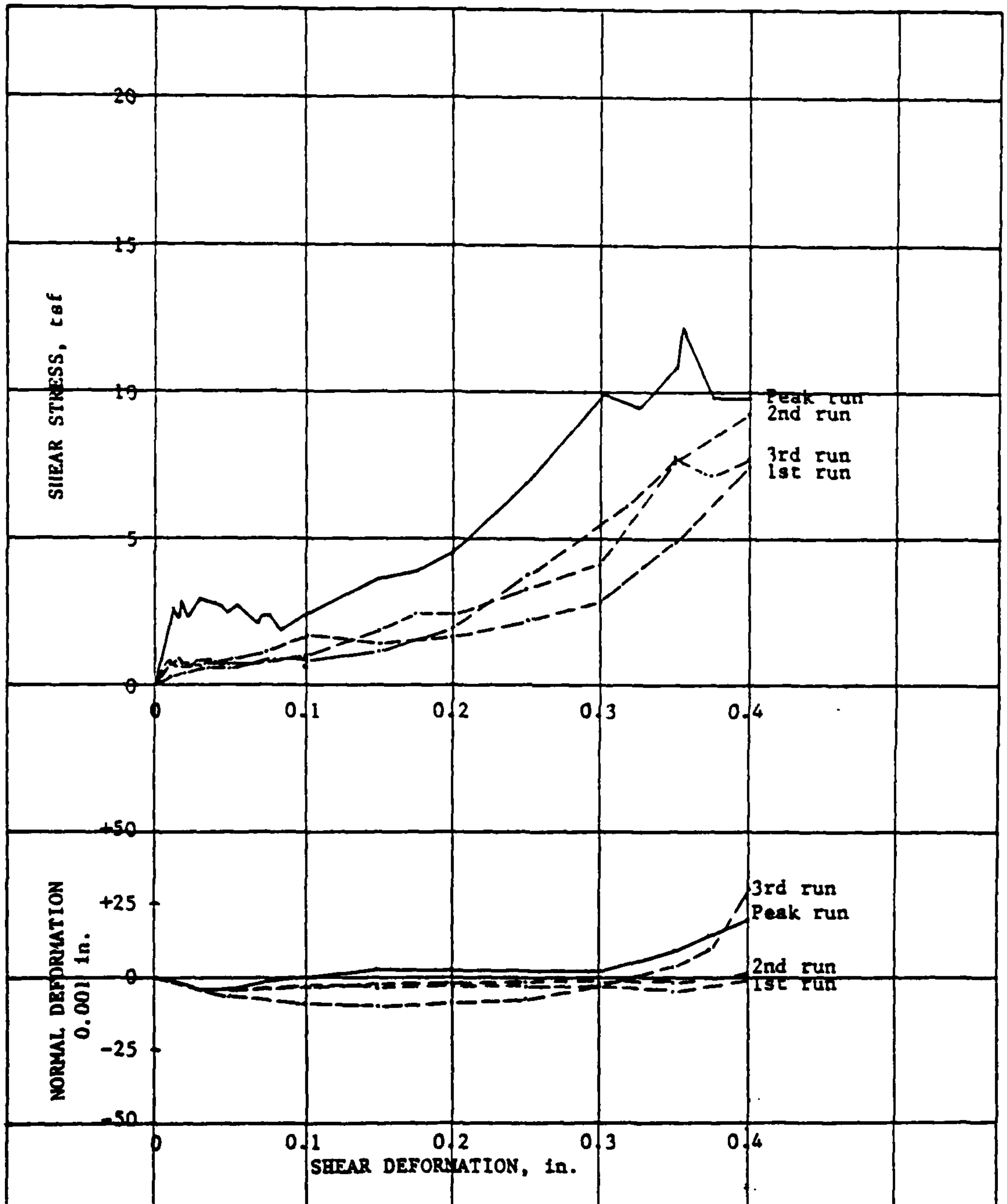
DIRECT SHEAR - CLAY FILLING IN LIMESTONE
BORING DWES RB-2-79, EL 475.4, σ_n = 2.5 tsf



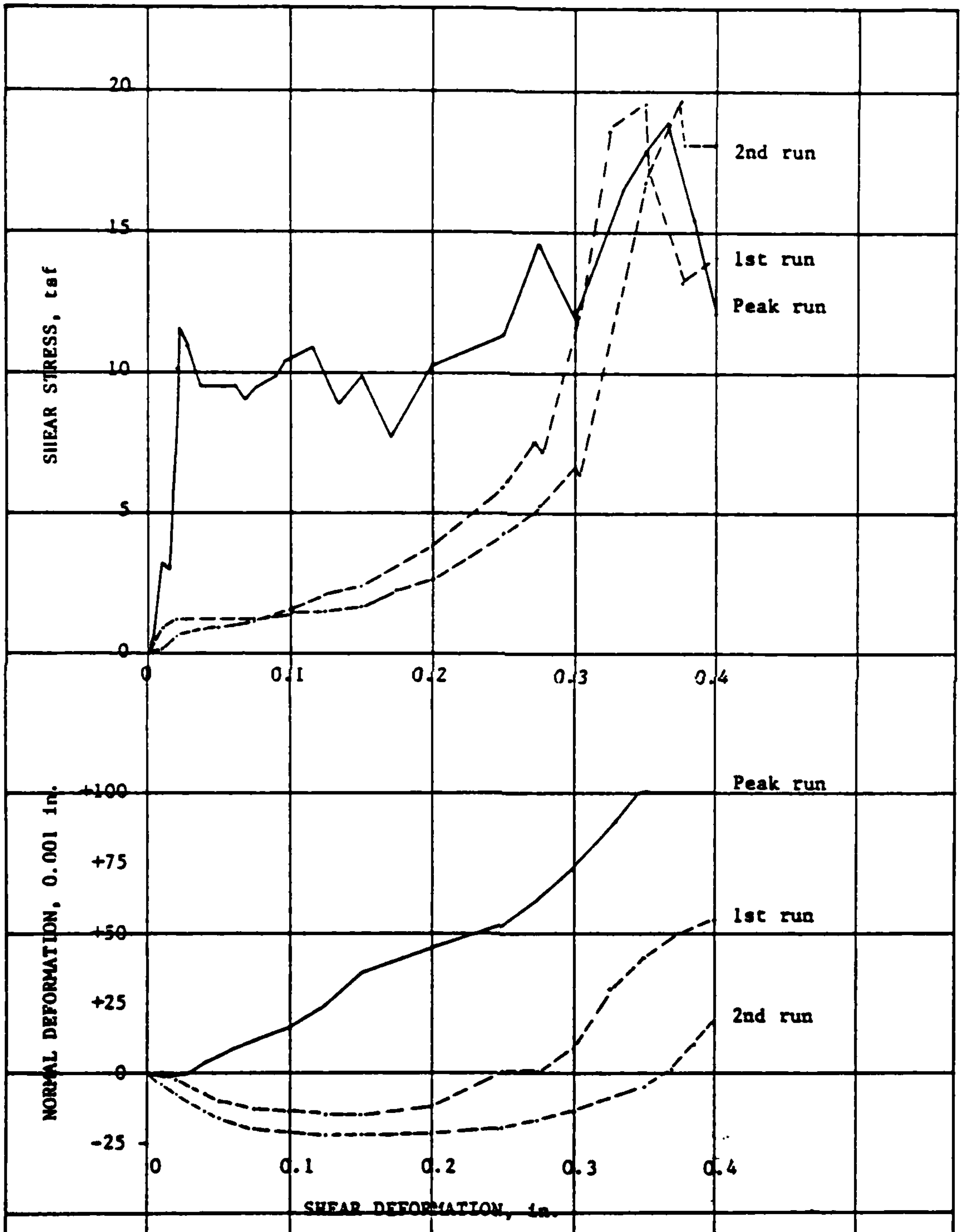
DIRECT SHEAR - CLAY FILLED PARTING IN LIMESTONE
BORING DIWES RB-2-79, EL 478.7, σ_n = 4.0 tsf



DIRECT SHEAR - CLAY FILLED PARTING ~~IN~~ LIMESTONE
BORING DIWES RB-5-79, EL 481.5, σ_n = 8.0 tsf

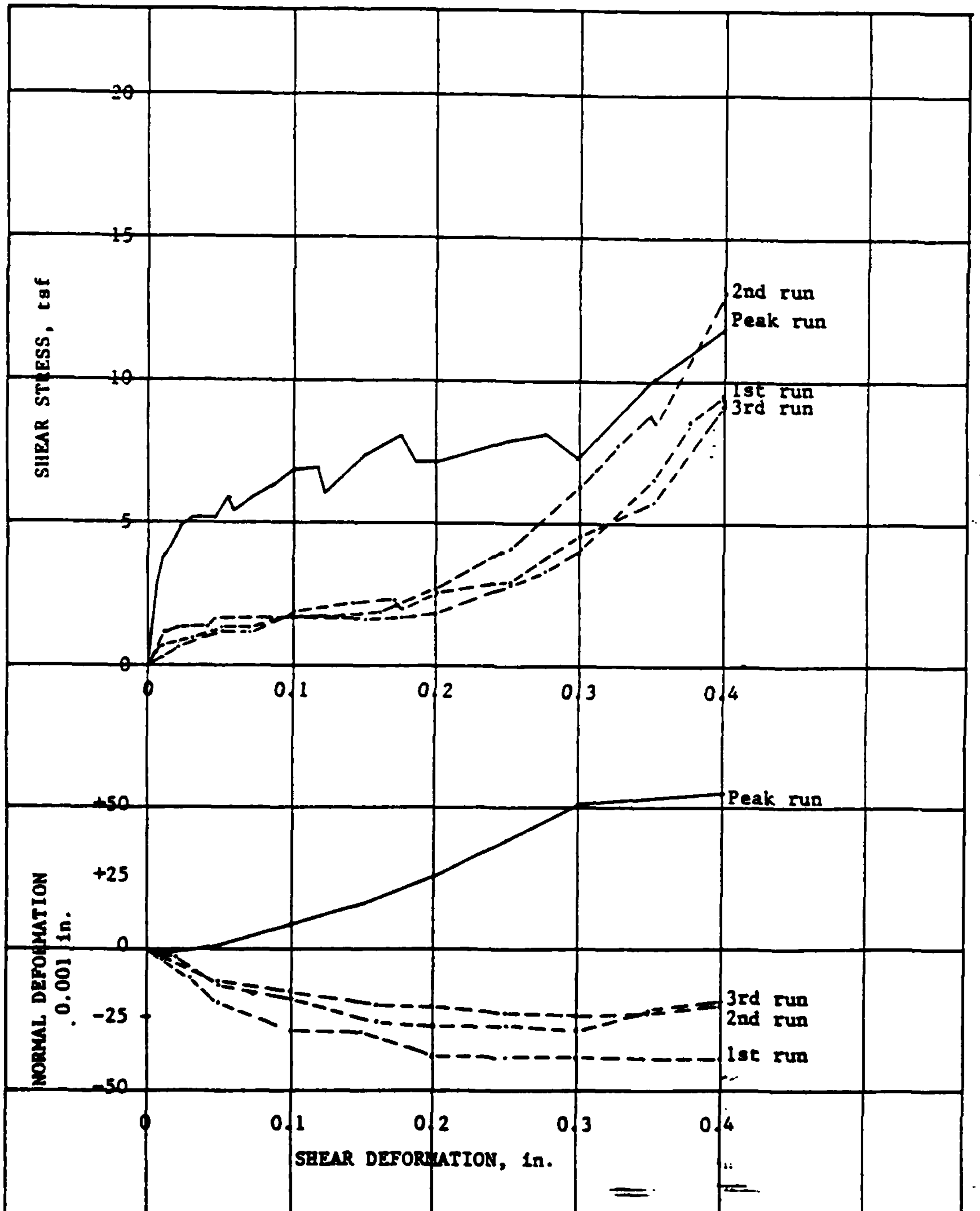


DIRECT SHEAR - SHALE/CLAY FILLED PARTING IN LIMESTONE
BORING DI WES RB-2-79, EL 480.7 ft, σ_n = 1.5 tsf

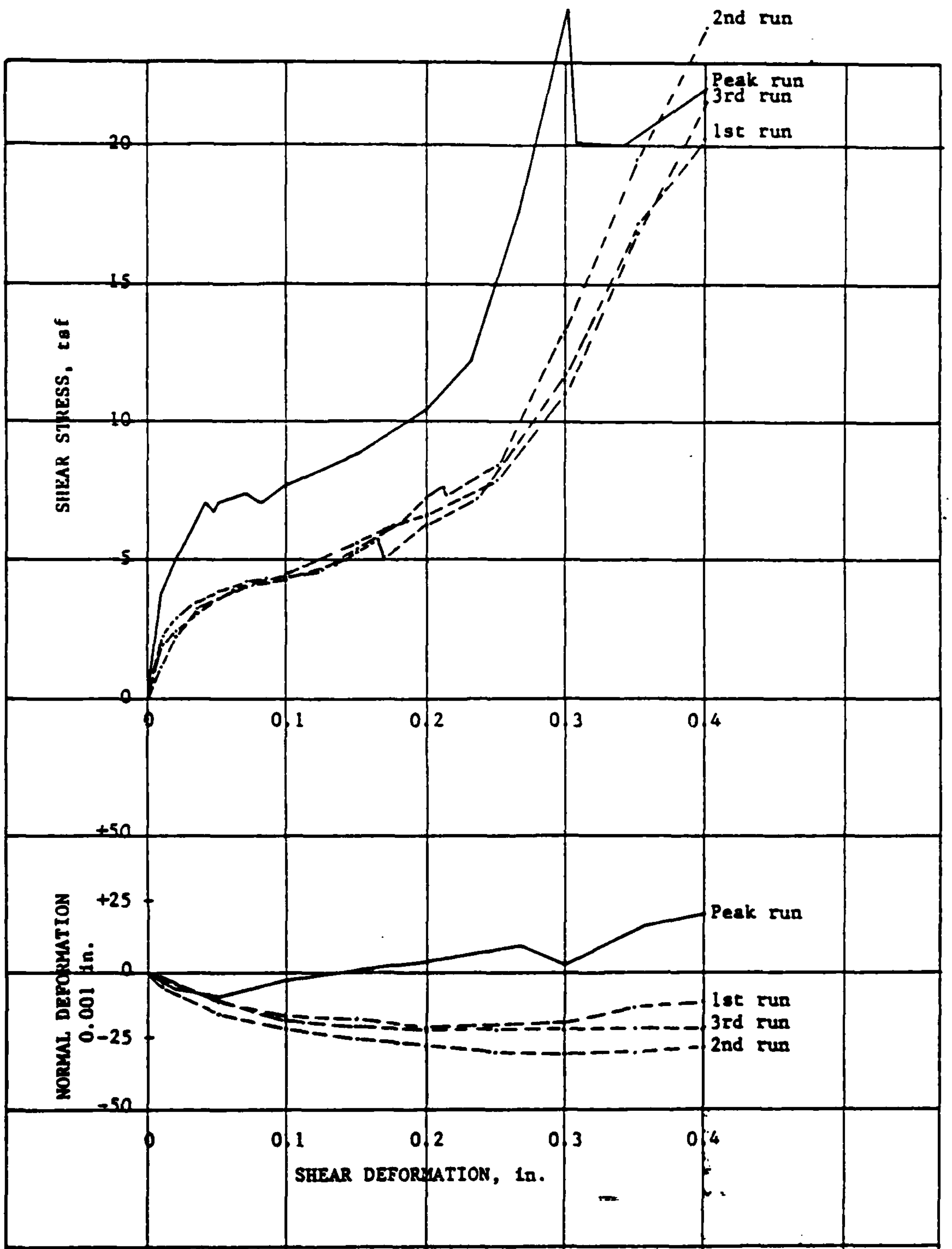


DIRECT SHEAR - SHALE/CLAY FILLED PARTING IN LIMESTONE
BORING DI WES RB-2-79, EL 480.3 ft, σ_n = 2.5 tsf

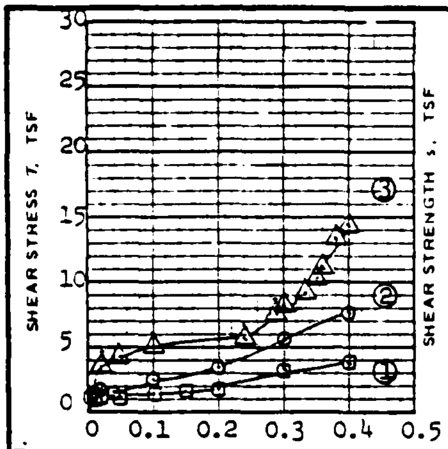
PLATE 45

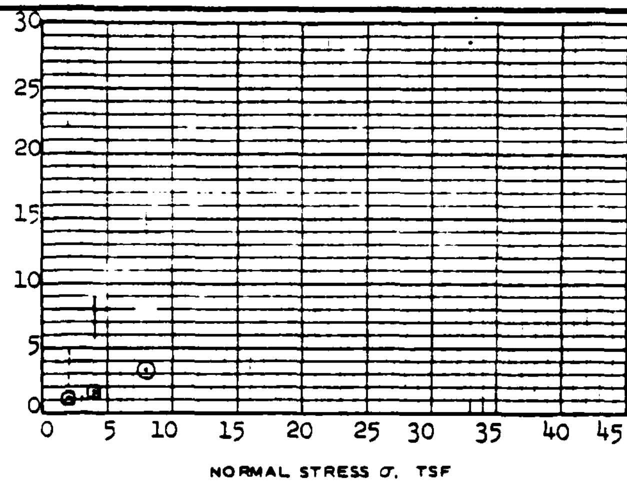


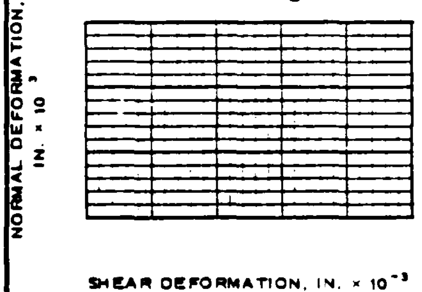
DIRECT SHEAR - SHALE/CLAY FILLED PARTING IN LIMESTONE
BORING DI WES RB-2-79, EL 477.9 ft, σ_n = 4.0 tsf



DIRECT SHEAR - SHALE/CLAY FILLED PARTING IN LIMESTONE
BORING DI WES RB-4-79, EL 483.1 ft, σ_n = 8.0 tsf







SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE	
ϕ =			
TAN ϕ =			
c =			TSF

TEST NO.		1	2	3			
WET DENSITY, PCF	γ_m	150.4	161.1	165.6			
WATER CONTENT	w	8.1 %	6.7 %	4.5 %	%	%	%
RATE OF SHEAR INCREASED AT		0.020	0.060	NO INCR.			
NORMAL STRESS, TSF	σ	2.0	4.0	8.0			
MAXIMUM SHEAR STRESS, TSF	τ_f	1.5	1.6	3.4			
TIME TO FAILURE, MINUTES	t_f	7	7	3			
ULTIMATE SHEAR STRESS, TSF	τ_r	1.02	1.62	14.5			
INITIAL DIAMETER, IN.	Do	5.74	5.74	5.74			
INITIAL HEIGHT, IN.	Ho						

DESCRIPTION OF MATERIAL PRECUT ROCK ON ROCK, GRAY TO BROWN SHALE

REMARKS
O MAXIMUM SHEAR STRESS
□ ULTIMATE SHEAR STRESS

PROJECT DRESDEN ISLAND LOCK AND DAM

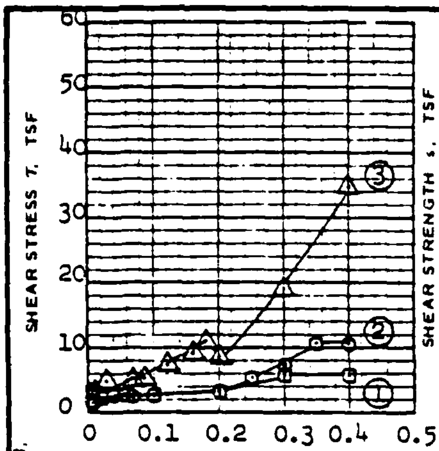
AREA

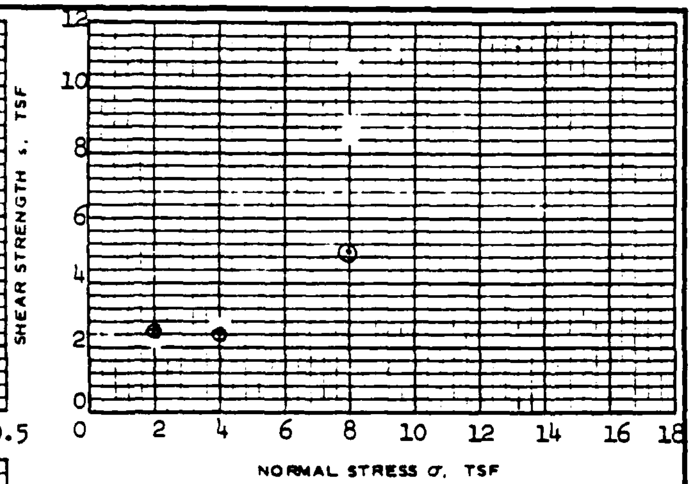
BORING NO. GW-5 SAMPLE NO.

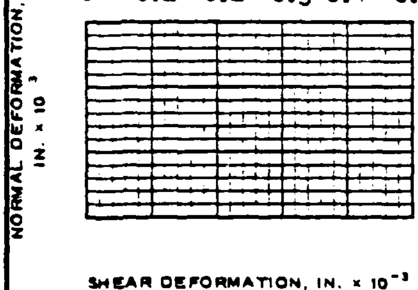
DEPTH EL 30.9 DATE 23JUL77

RCH DIRECT SHEAR TEST REPORT (ROCK)

WES FORM APR 73 1490 EDITION OF JUN 65 IS OBSOLETE







SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE
ϕ =		
TAN ϕ =		
c =		TSF

TEST NO.		1	2	3			
WET DENSITY, PCF	γ_m						
WATER CONTENT	w	%	%	%	%	%	%
RATE OF SHEAR INCREASED AT		0.020	0.064	0.20			
NORMAL STRESS, TSF	σ	2.0	4.0	8.0			
MAXIMUM SHEAR STRESS, TSF	τ_f	2.5	2.4	5.0			
TIME TO FAILURE, MINUTES	t_f	6	16	30			
ULTIMATE SHEAR STRESS, TSF	τ_r						
INITIAL DIAMETER, IN.	Do	5.78	5.78	5.78			
INITIAL HEIGHT, IN.	Ho						

DESCRIPTION OF MATERIAL PRECUT ROCK ON ROCK, GRAY TO BROWN SHALE

REMARKS
o MAXIMUM SHEAR STRESS
□ ULTIMATE SHEAR STRESS

PROJECT DRESDEN ISLAND LOCK AND DAM

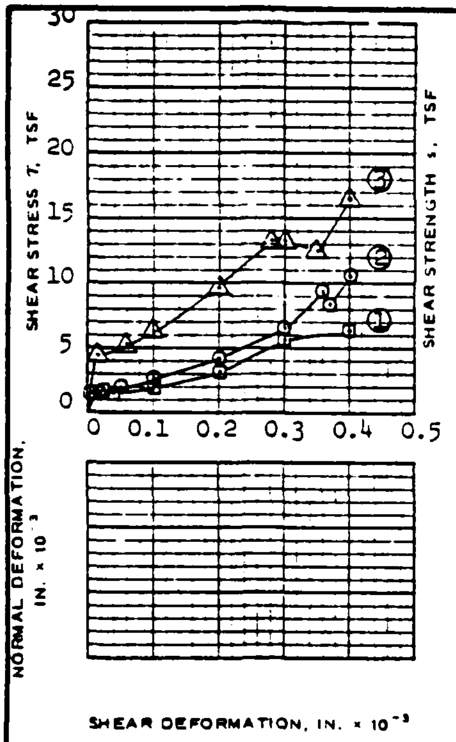
AREA

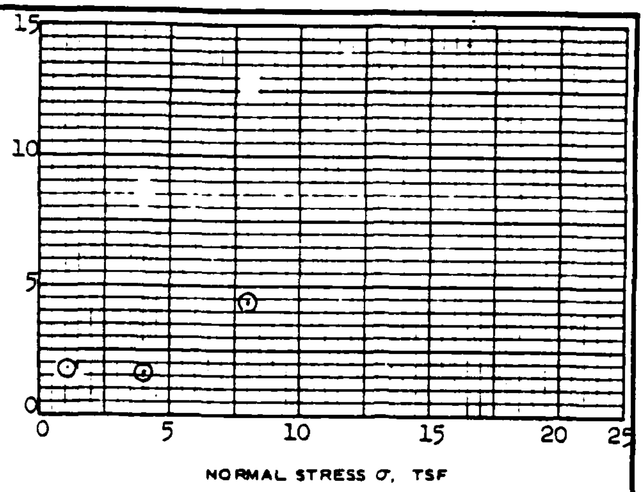
BORING NO. GW-5 SAMPLE NO.

DEPTH EL. 32.3 DATE 22JUL77

RCH DIRECT SHEAR TEST REPORT (ROCK)

WES FORM APR 75 1490 EDITION OF JUN 65 IS OBSOLETE





SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE	
φ =			
TAN φ =			
c =			TSF

TEST NO.		1	2	3			
WET DENSITY, PCF	γ_m						
WATER CONTENT	w	%	%	%	%	%	%
RATE OF SHEAR INCREASED AT		0.04	NO INCR.	NO INCR.			
NORMAL STRESS, TSF	σ	2.0	4.0	8.0			
MAXIMUM SHEAR STRESS, TSF	τ_f	1.5	1.6	4.4			
TIME TO FAILURE, MINUTES	t_f	7	32	18			
ULTIMATE SHEAR STRESS, TSF	τ_r						
INITIAL DIAMETER, IN.	Do	5.85	5.85	5.85			
INITIAL HEIGHT, IN.	Ho						

DESCRIPTION OF MATERIAL PRECUT ROCK ON ROCK, GRAY TO BROWN SHALE

REMARKS

o MAXIMUM SHEAR STRESS

▫ ULTIMATE SHEAR STRESS

PROJECT DRESDEN ISLAND LOCK AND DAM

AREA

BORING NO. GW-5

SAMPLE NO.

DEPTH EL. 33.9

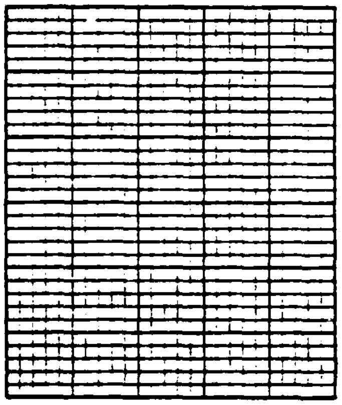
DATE

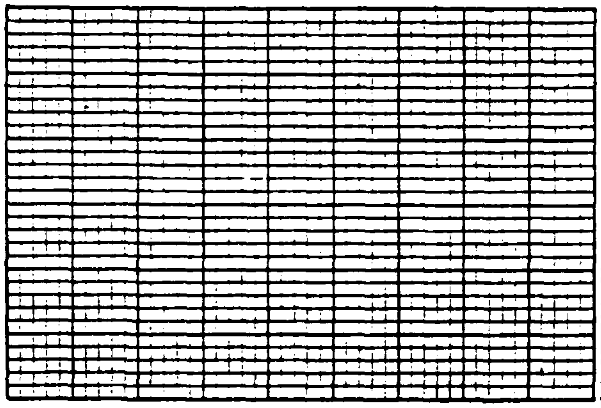
RCH DIRECT SHEAR TEST REPORT (ROCK)

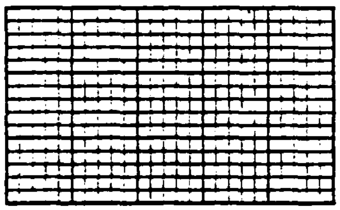
WES FORM APR 78 1490 EDITION OF JUN 65 IS OBSOLETE

PLATE 50

SHEET NO.







SHEAR STRENGTH PARAMETERS

	MAXIMUM	ULTIMATE	
ϕ =			
TAN ϕ =			
c =			TSF

TEST NO. Boring No. / Elevation, ft		RB-2-79 474.9	D-35-79 462.8	RB-4-79 476.9	RB-5-79 475.8		
WET DENSITY, PCF	γ_d	—	133.0	—	—		
WATER CONTENT	w	— %	12.8%	— %	— %	%	%
NORMAL STRESS, TSF	σ	1.5	2.0	2.5	4.0		
MAXIMUM SHEAR STRESS, TSF	τ_f	0.582	0.737	1.520	1.760		
TIME TO FAILURE, MINUTES	t_f						
ULTIMATE SHEAR STRESS, TSF	τ_r						
INITIAL DIAMETER, IN.	Do						
INITIAL HEIGHT, IN.	Ho						

DESCRIPTION OF MATERIAL S-Test Green Clay (Plastic)

REMARKS

PROJECT Dresden Island Lock and Dam
Compliance & Scour Detection

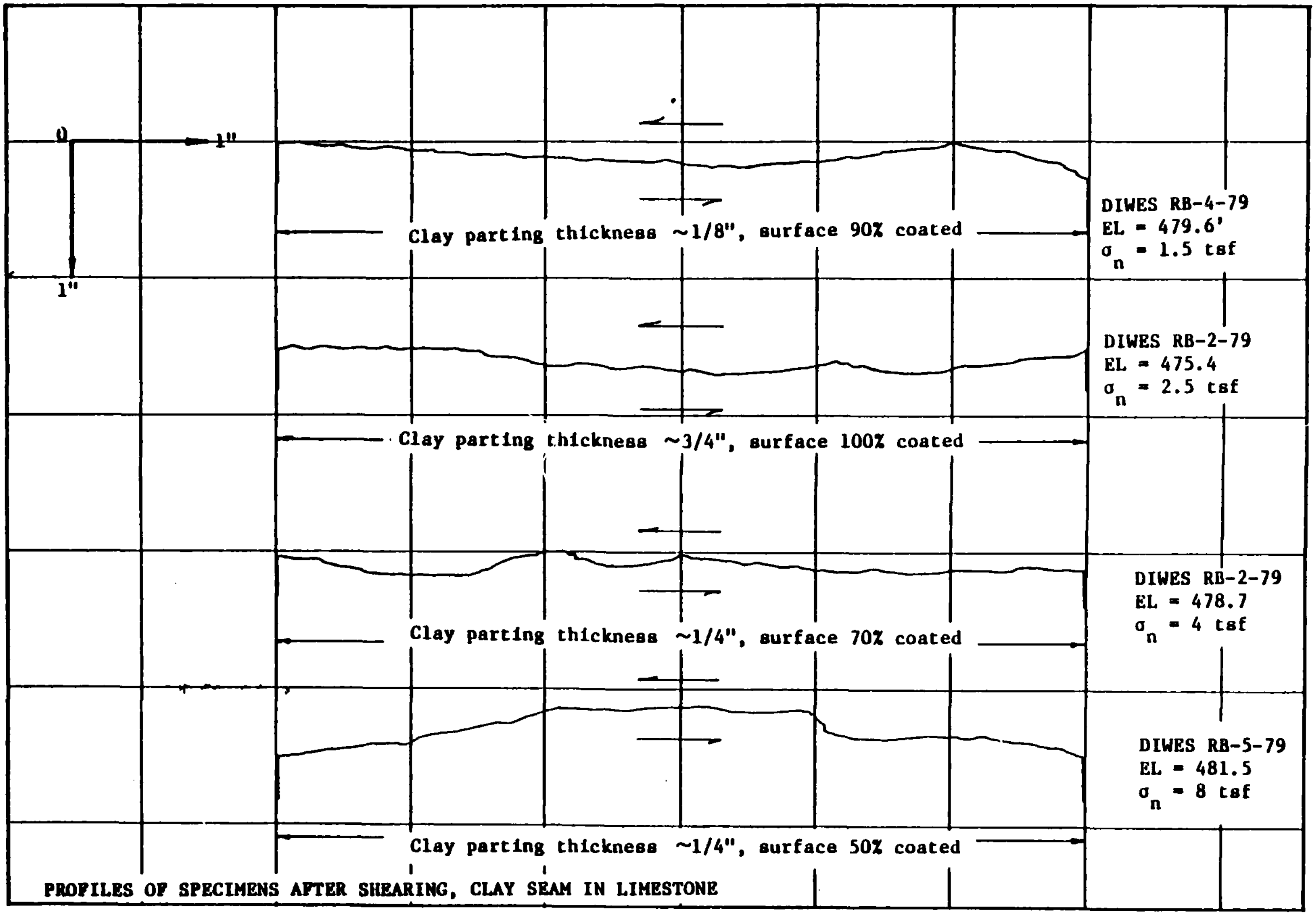
AREA

BORING NO. See Test No. SAMPLE NO.

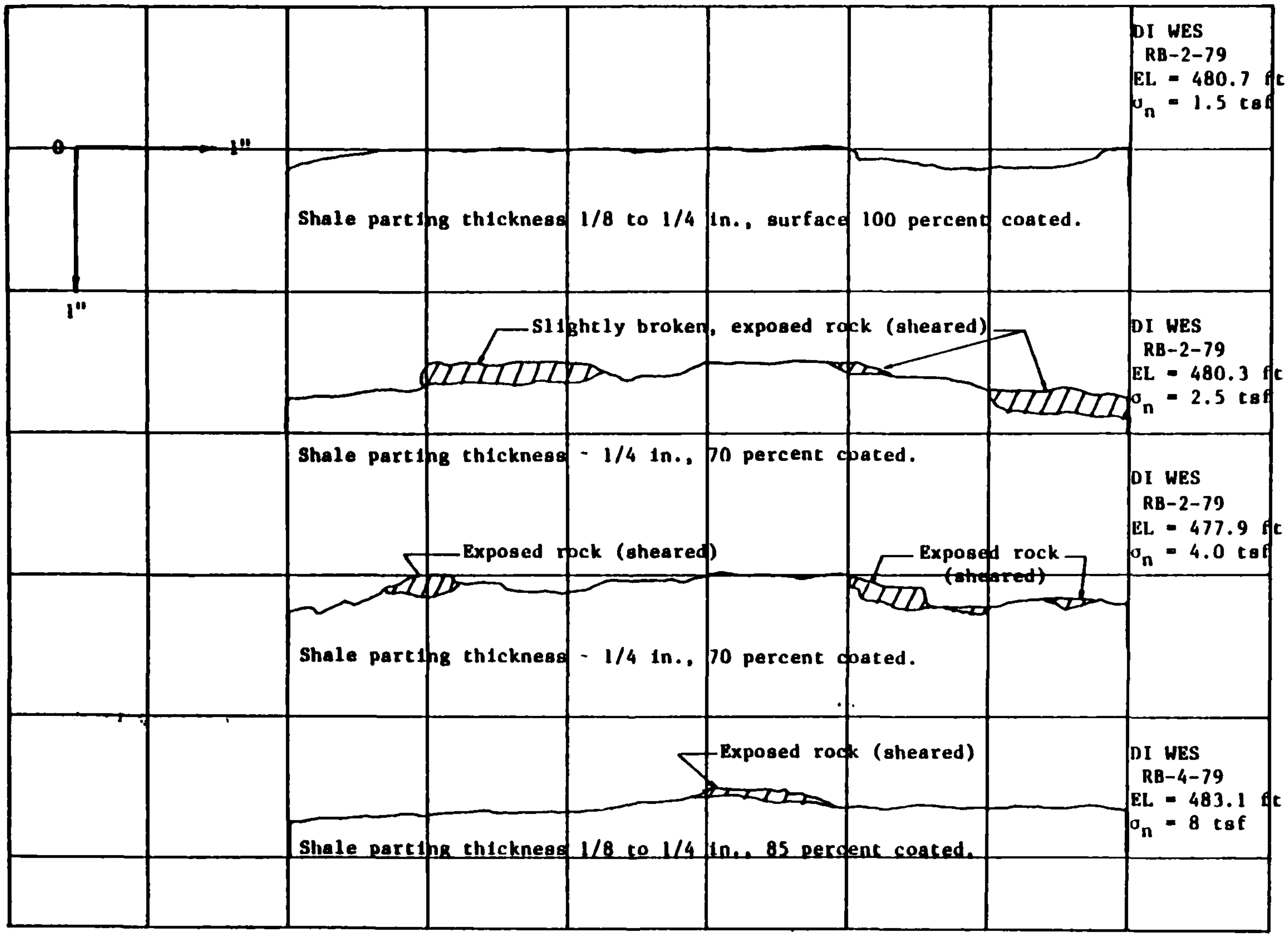
DEPTH / EL See Test No. DATE 14 Aug 80

DIRECT SHEAR TEST REPORT (ROCK)

WES FORM APR 76 1490 EDITION OF JUN 65 IS OBSOLETE



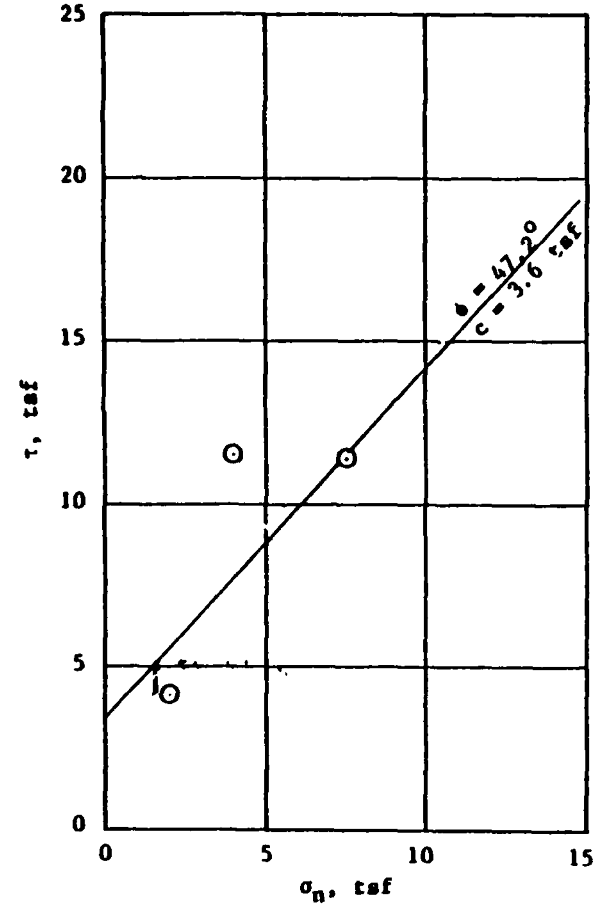
PROFILES OF SPECIMENS AFTER SHEARING, CLAY SEAM IN LIMESTONE



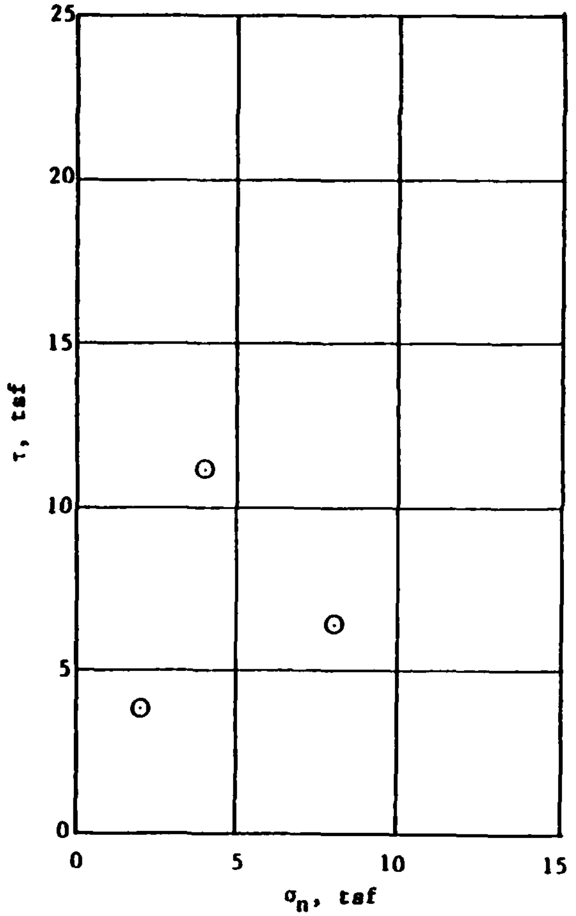
PROFILES OF SPECIMENS AFTER SHEARING, SHALE/CLAY SEAMS IN LIMESTONE

DRESDEN ISLAND LOCK AND DAM COMPLIANCE PHASE SCOUR DETECTION

PEAK SHEAR STRENGTH

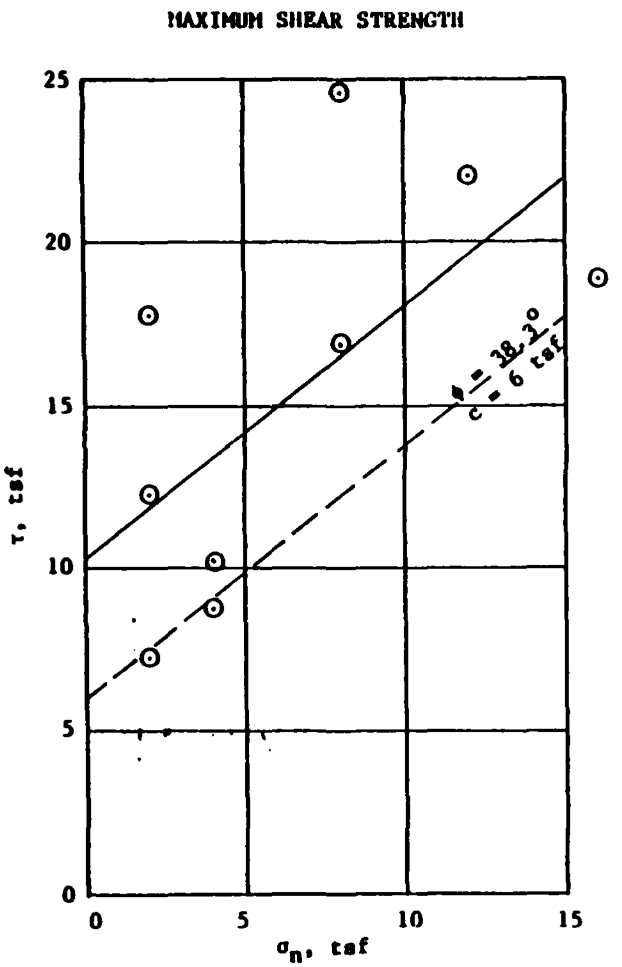


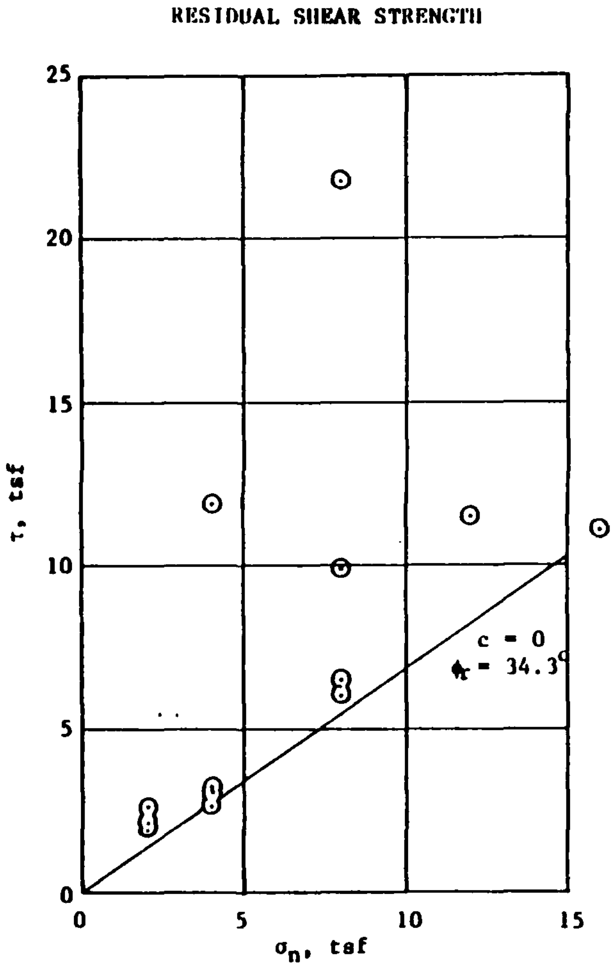
ULTIMATE SHEAR STRENGTH



Failure envelope for intact gray to brown shale, direct shear.

DRESDEN ISLAND LOCK AND DAM COMPLIANCE PHASE SCOUR DETECTION

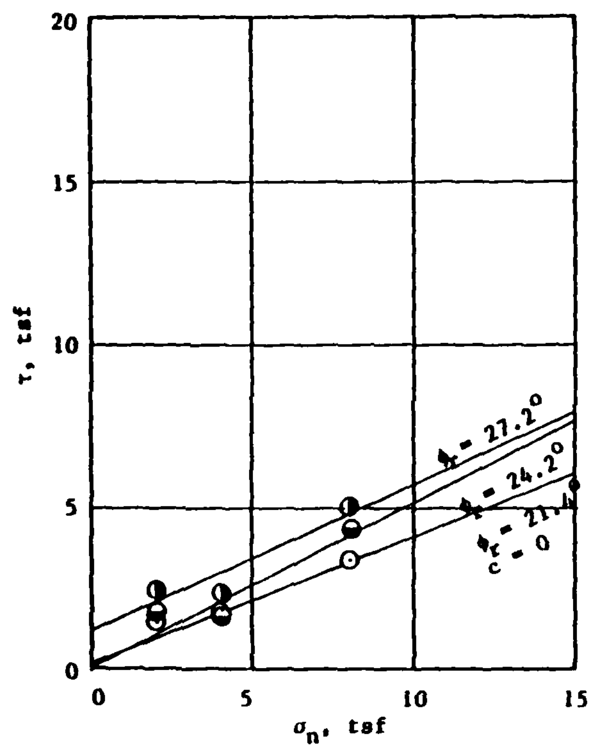




Failure envelope for intact specimens, green shale, direct shear.

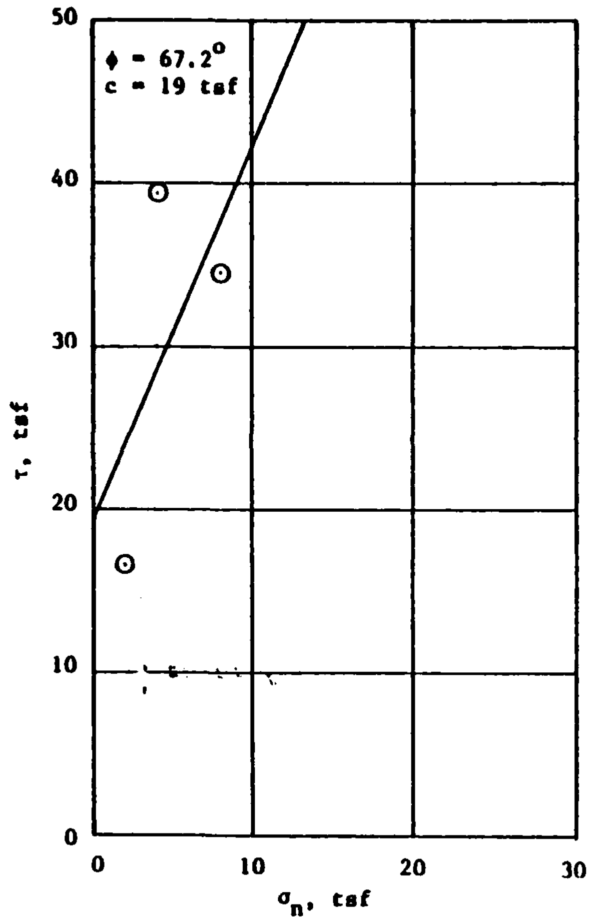
DRESDEN ISLAND LOCK AND DAM, MAJOR REHAB PHASE

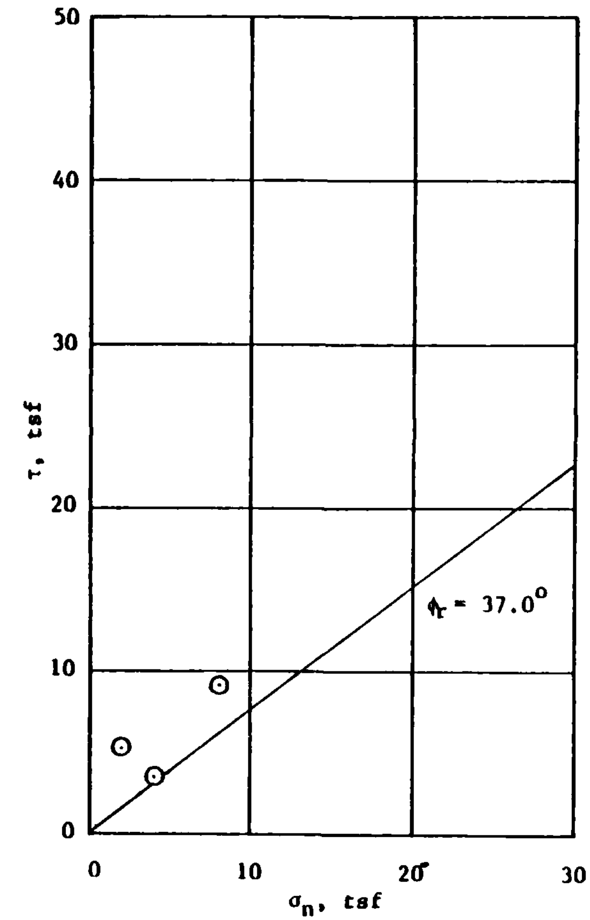
RESIDUAL SHEAR STRENGTH



Failure envelope for precut, rock-on-rock, gray to brown shale, multistage shear.

DRESDEN ISLAND LOCK AND DAM COMPLIANCE PHASE SCOUR DETECTION

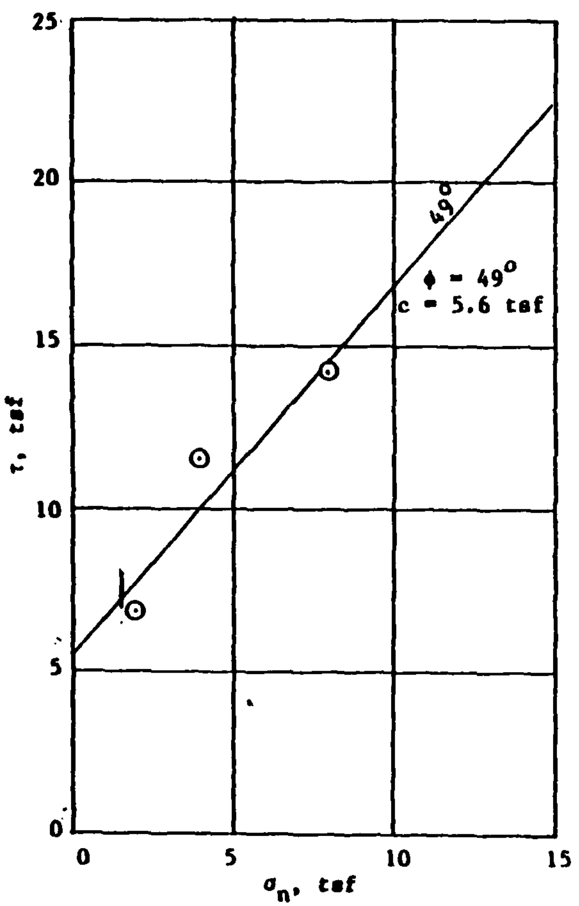




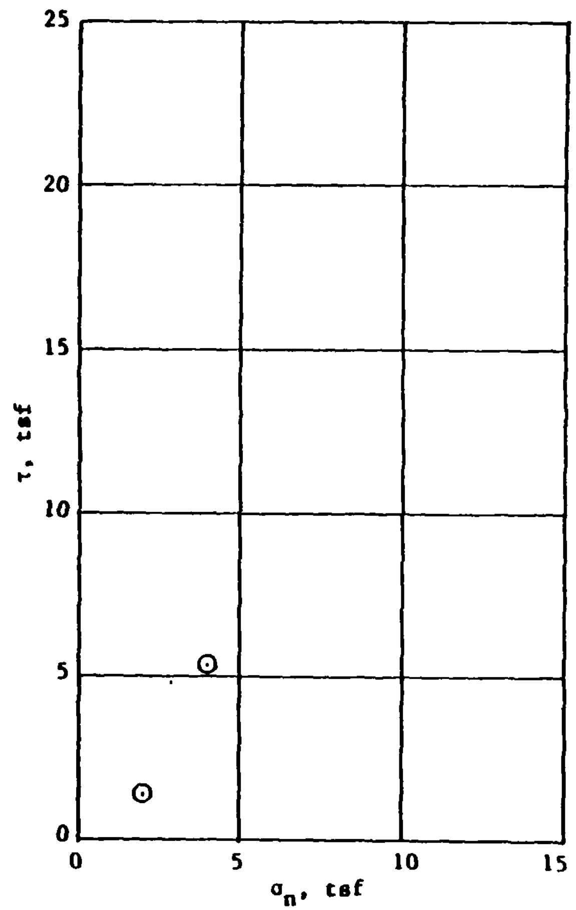
Failure envelope for concrete on rock, limestone, direct shear.

DRESDEN ISLAND LOCK AND DAM COMPLIANCE PHASE, SCOUR DETECTION

MAXIMUM SHEAR STRENGTH



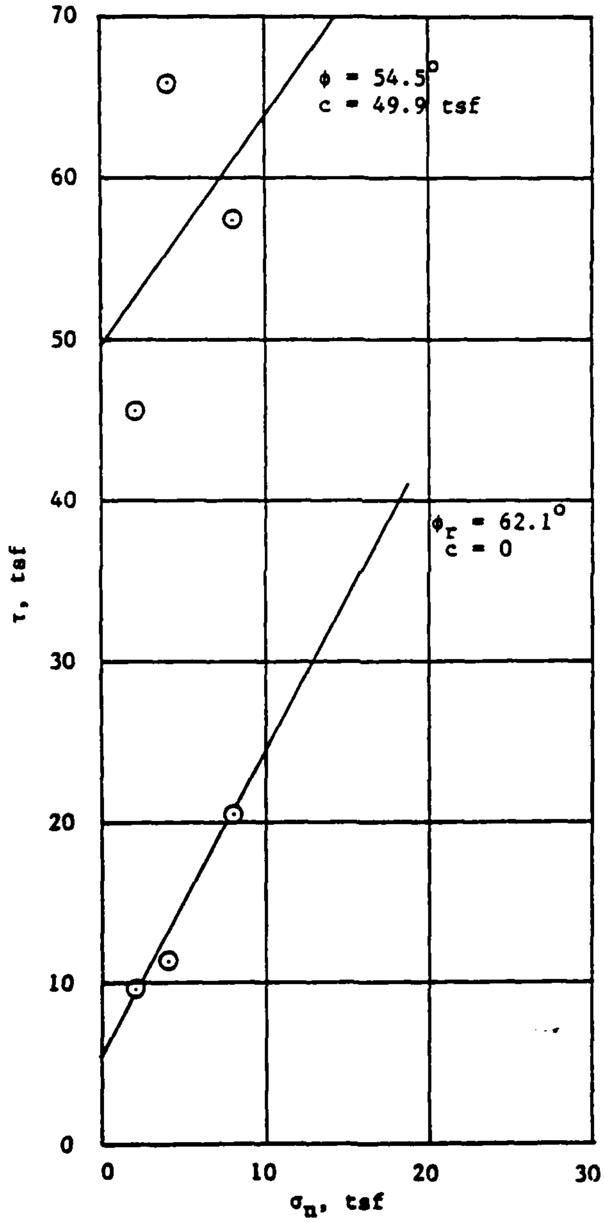
ULTIMATE SHEAR STRENGTH



Failure envelope for concrete on rock, gray to brown shale, direct shear.

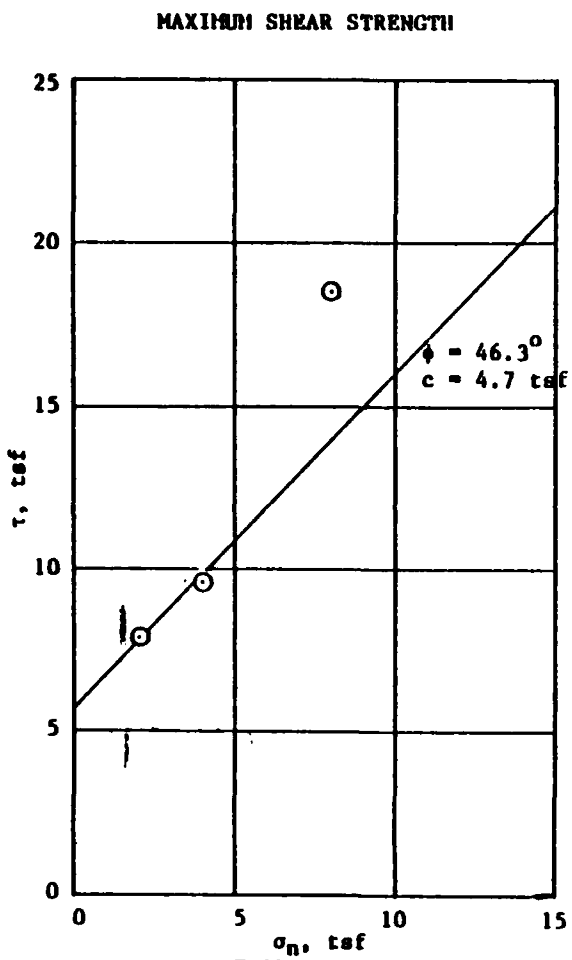
DRESDEN ISLAND LOCK AND DAM COMPLIANCE PHASE SCOUR DETECTION

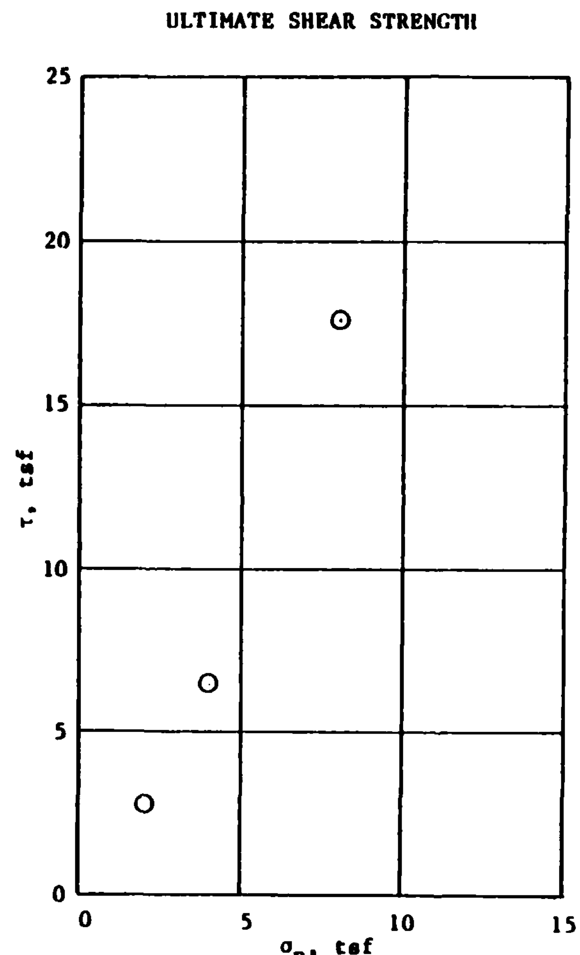
MAXIMUM SHEAR STRENGTH/ULTIMATE SHEAR STRENGTH



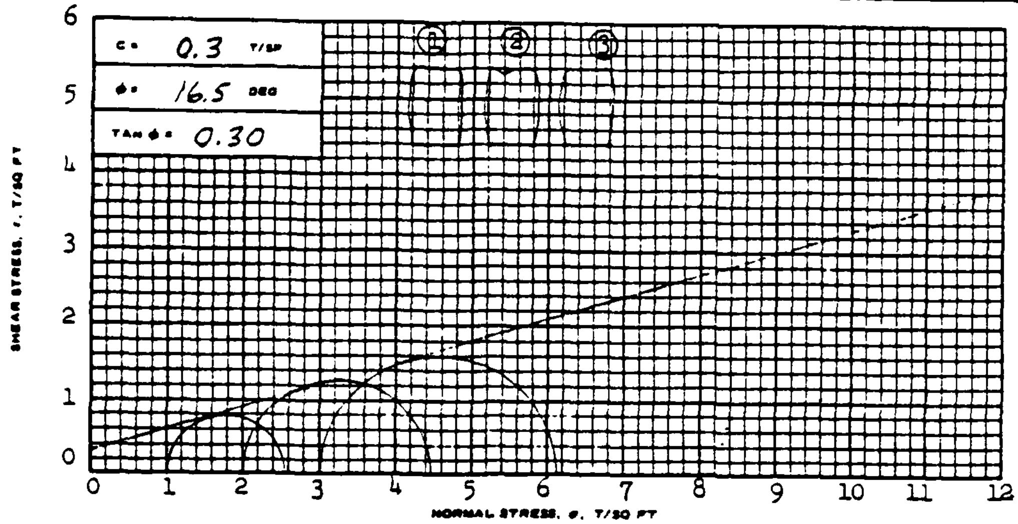
Failure envelope for cross bed, limestone, direct shear.

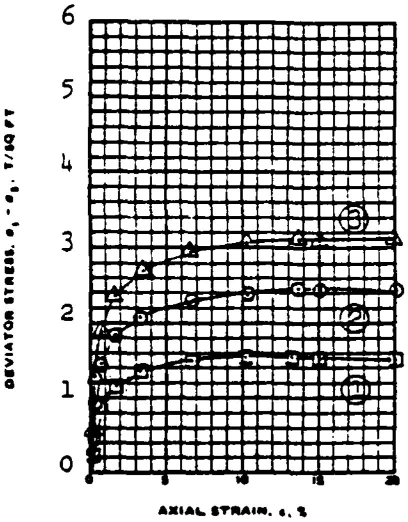
DRESDEN ISLAND LOCK AND DAM COMPLIANCE PHASE SCOUR DETECTION





Failure envelope for cross-bed, gray to brown shale, direct shear.





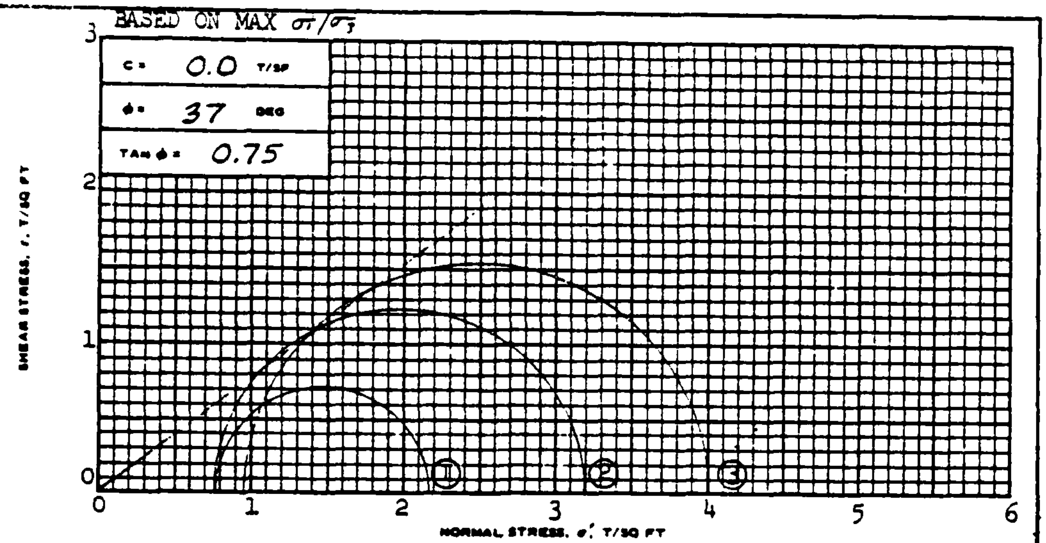
SPECIMEN NO.			1	2	3	
INITIAL	WATER CONTENT, %	w_o	33.9	33.5	35.4	
	DRY DENSITY LB/CU FT	γ_{d_o}	84.2	84.7	81.8	
	SATURATION, %	S_o	91.4	91.3	90.2	
	VOID RATIO	e_o	1.00	0.991	1.06	
BEFORE SHEAR	WATER CONTENT, %	w_c	35.7	31.8	31.6	
	DRY DENSITY LB/CU FT	γ_{d_c}	88.5	92.7	92.2	
	SATURATION, %	S_c	100+	100+	100+	
	VOID RATIO	e_c	0.905	0.819	0.829	
	FINAL BACK PRESSURE, T/SQ FT	u_o	7.20	7.20	7.20	
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.0	2.0	3.0	
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	1.56	2.43	3.11	
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_f	964	978	986	
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$				
INITIAL DIAMETER, IN.		D_o	1.42	1.42	1.41	
INITIAL HEIGHT, IN.		H_o	3.00	3.00	3.00	

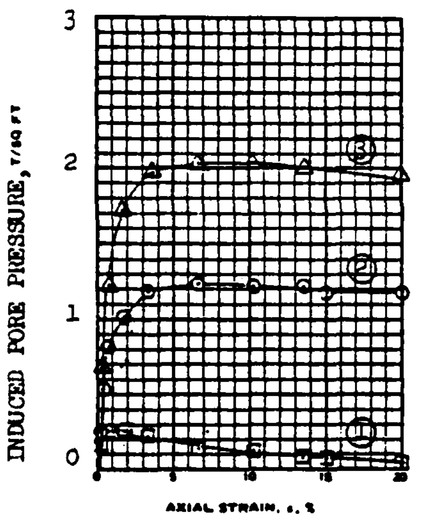
CONTROLLED- STRAIN TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH), VERY DARK GRAY;FINE ORGANIC MATTER

LL	PL	PI	G_s 2.70	TYPE OF SPECIMEN UNDISTURBED	TYPE OF TEST R
REMARKS: SEE ATTACHED SHEET FOR EFFECTIVE VALUES.				PROJECT DRESDEN ISLAND LOCK & DAM	
				BORING NO. E-1	SAMPLE NO.
				DEPTH/ELEV 15.0-15.4/[illegible]	
				LABORATORY USAEWES	DATE 7 APR 78
SHEET 1 OF 2.				JMS TRIAXIAL COMPRESSION TEST REPORT	

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT (EM 1110-2-1906)





SPECIMEN NO.						
INITIAL	WATER CONTENT, %	w_o				
	DRY DENSITY LB/CU FT	γ_{d_o}				
	SATURATION, %	S_o				
	VOID RATIO	e_o				
BEFORE SHEAR	WATER CONTENT, %	w_c				
	DRY DENSITY LB/CU FT	γ_{d_c}				
	SATURATION, %	S_c				
	VOID RATIO	e_c				
	FINAL BACK PRESSURE, T/SQ FT	u_o				
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3				
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$				
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_f				
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$				
INITIAL DIAMETER, IN.		D_o				
INITIAL HEIGHT, IN.		H_o				

CONTROLLED- TEST

DESCRIPTION OF SPECIMENS

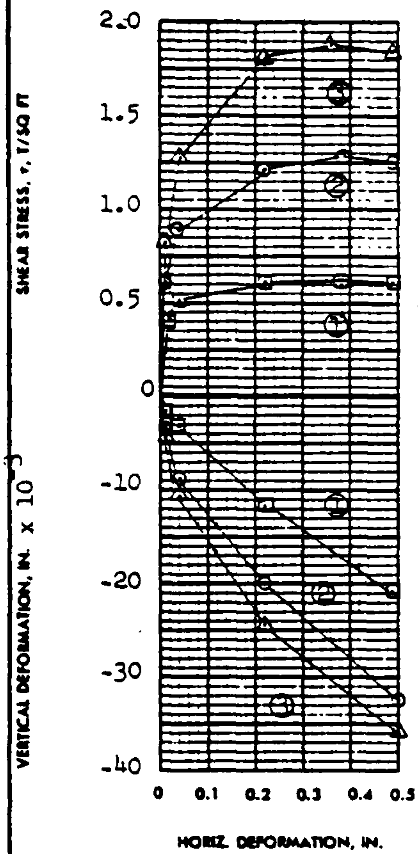
LL	PL	PI	Gs	TYPE OF SPECIMEN	TYPE OF TEST

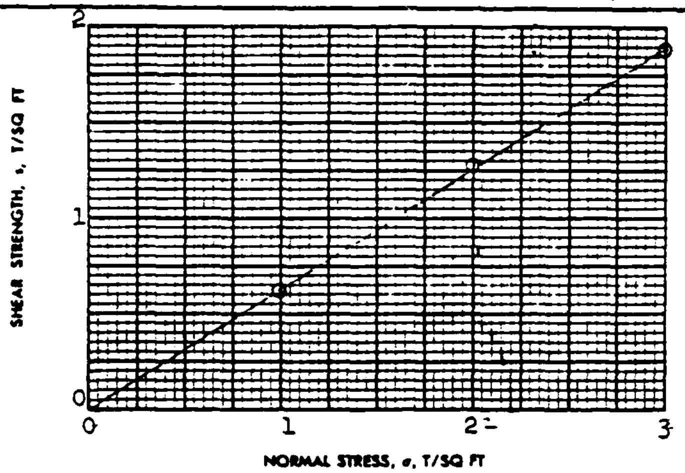
REMARKS

PROJECT	DRESDEN ISLAND LOCK & DAM
BORING NO.	E-1
SAMPLE NO.	
DEPTH/ELEV	15.0-15.4
LABORATORY	USAEWES
DATE	7 Apr 78

JMS TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 2089 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT (EM 1110-2-1906)





SHEAR STRENGTH PARAMETERS

φ' = 32°

TAN φ' = 0.62

c' = 0.0 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

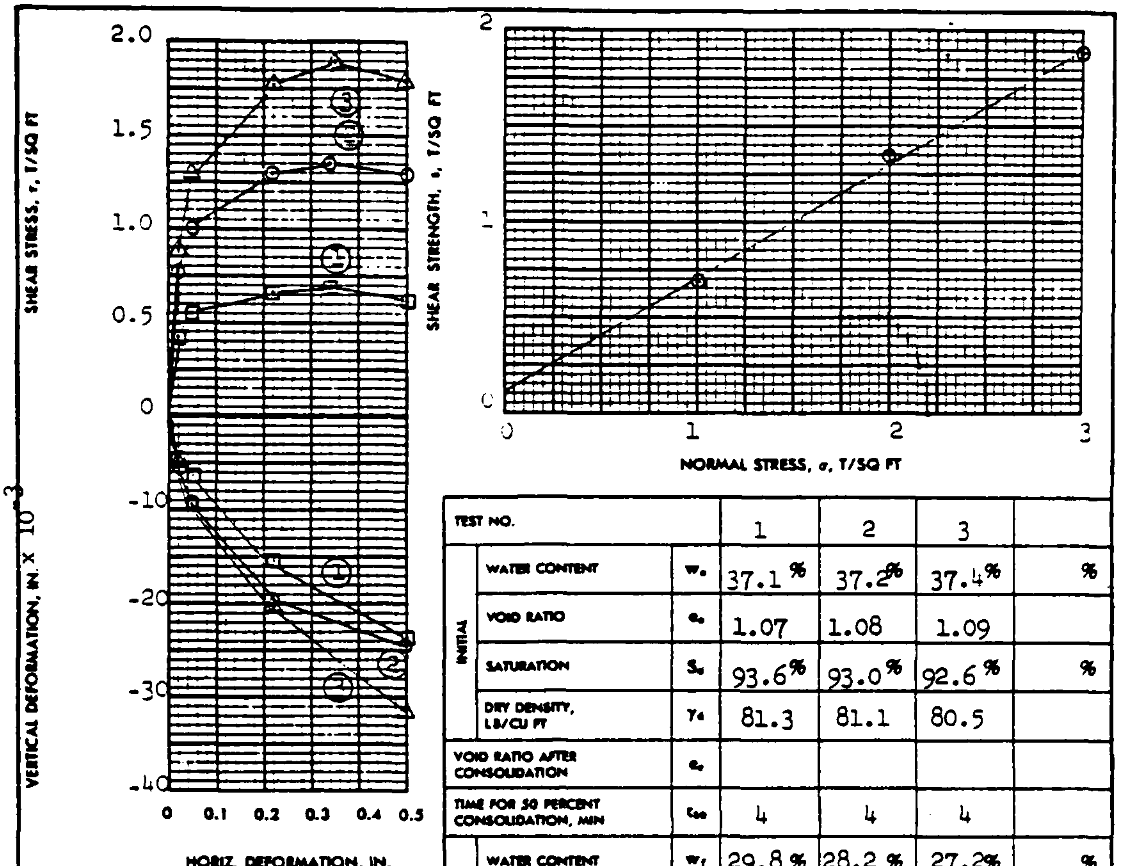
TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	34.6%	34.6%	34.4 %	%
	VOID RATIO	e_o	1.02	1.04	1.02	
	SATURATION	S_o	91.6%	89.8%	91.1 %	%
	DRY DENSITY, LB/CU FT	γ_d	83.6	82.8	83.4	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	2	2	2	
FINAL	WATER CONTENT	w_f	31.4%	28.8%	26.9%	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	1.0	2.0	3.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.63	1.29	1.89	
ACTUAL TIME TO FAILURE, MIN		t_f	2220	2250	2100	
RATE OF STRAIN, IN./MIN			.00018	.00018	.00018	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN UNDISTURBED			3.00 IN. SQUARE	0.544 IN. THICK
CLASSIFICATION PLASTIC CLAY(CH), VERY DARK GRAY;FINE ORGANIC MATTER				
LL 52	PL 28	PI 24		G_s 2.70

REMARKS

PROJECT	DRESDEN ISLAND LOCK AND DAM
AREA	
BORING NO. E-1	SAMPLE NO.
DEPTH EL. 15.4-15.9/494.1-494.6	DATE 21 FEB 78
EWG	DIRECT SHEAR TEST REPORT

ENG FORM 1 JUN 65 2092 (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE



SHEAR STRENGTH PARAMETERS

φ' = 31°

TAN φ' = 0.60

c' = 0.11 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	37.1 %	37.2%	37.4%	%
	VOID RATIO	e_o	1.07	1.08	1.09	
	SATURATION	S_o	93.6%	93.0 %	92.6 %	%
	DRY DENSITY, LB/CU FT	γ_d	81.3	81.1	80.5	
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	4	4	4	
FINAL	WATER CONTENT	w_f	29.8 %	28.2 %	27.2%	%
	VOID RATIO	e_f				
	SATURATION	S_f	%	%	%	%
NORMAL STRESS, T/SQ FT		σ	1.0	2.0	3.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.69	1.35	1.89	
ACTUAL TIME TO FAILURE, MIN		t_f	2010	1980	2040	
RATE OF STRAIN, IN./MIN			.00018	.00018	.00018	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN: UNDISTURBED — 3.00 IN. SQUARE — 0.544 IN. THICK

CLASSIFICATION: PLASTIC CLAY(CH), VERY DARK GRAY: FINE ORGANIC MATTER;TRACE FINE*

LL 56 | PL 29 | PI 27 | G_s 2.70

REMARKS: *GRAVEL

PROJECT: DRESDEN ISLAND LOCK & DAM

AREA:

BORING NO. E-1 | SAMPLE NO.

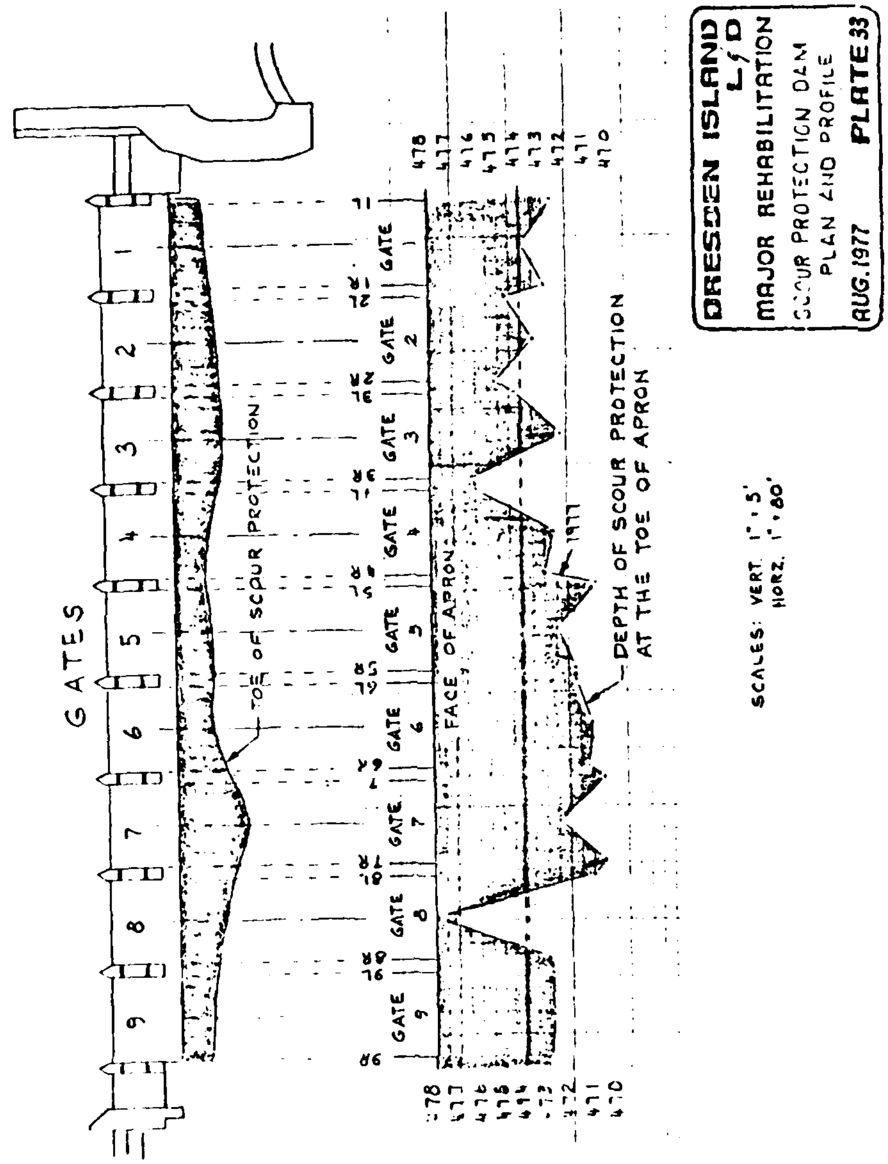
DEPTH / EL 13.6-14.0/496.4-496.0 | DATE 17 FEB 78

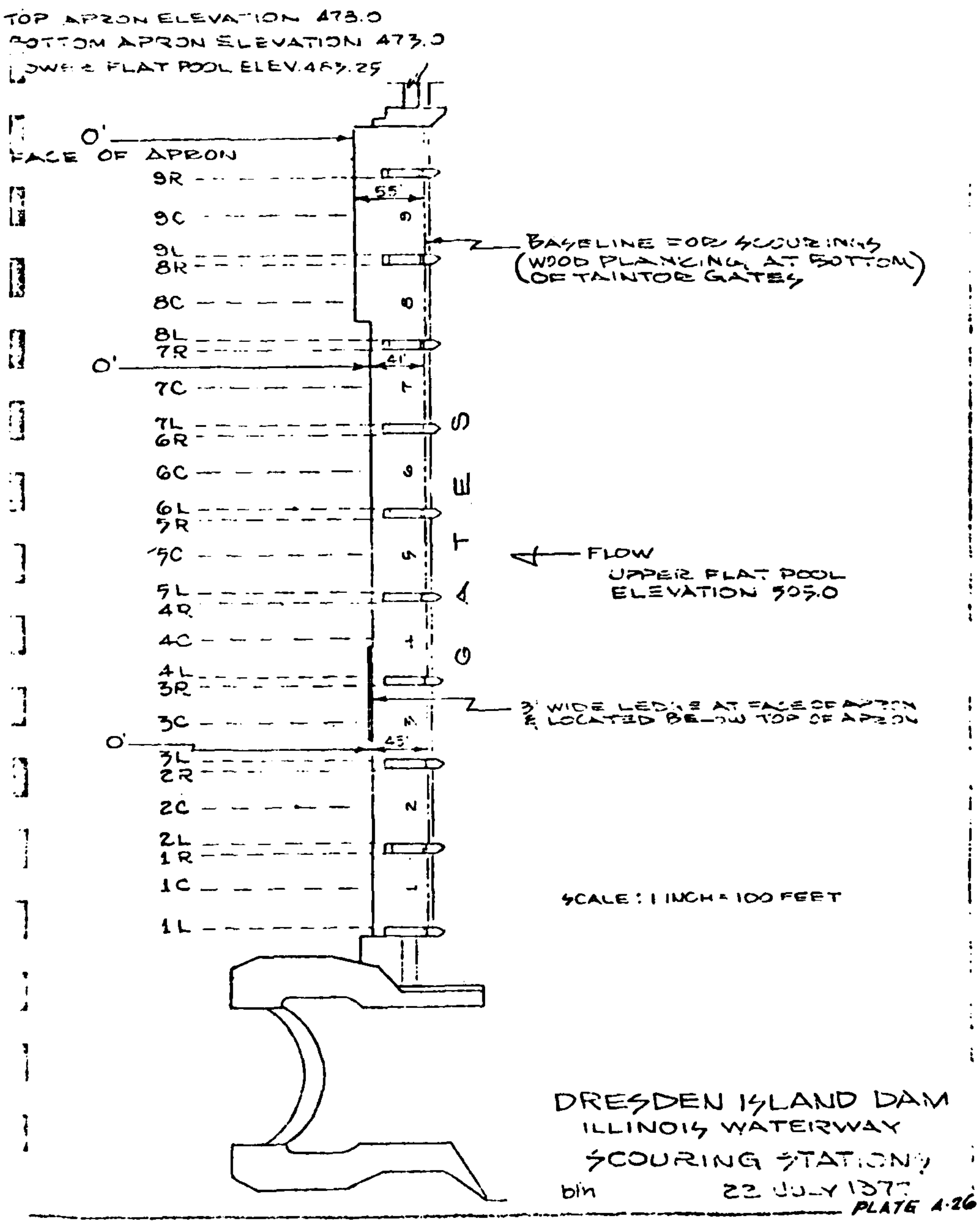
RCH — **DIRECT SHEAR TEST REPORT**

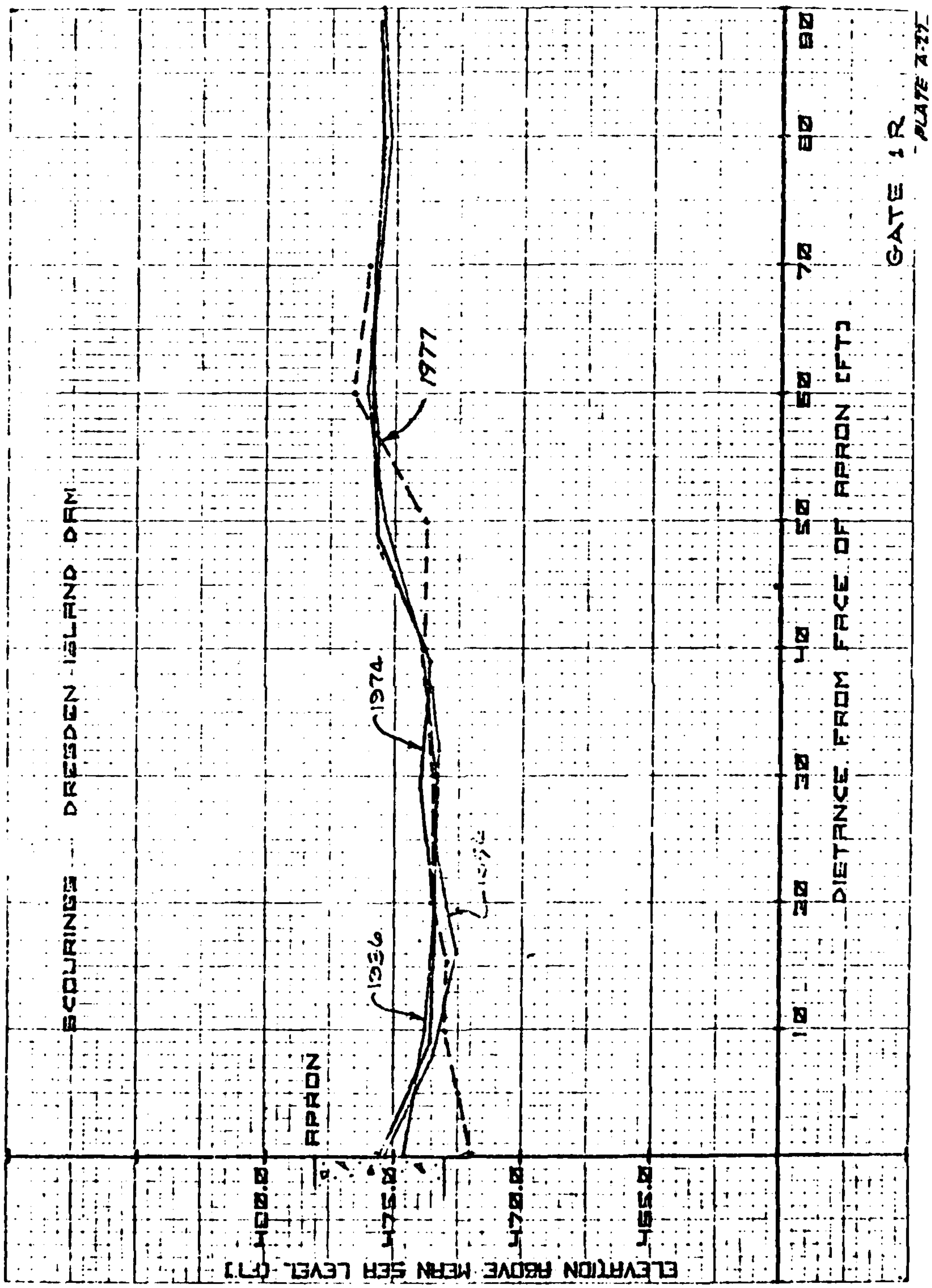
ENG FORM 2092, 1 JUN 68 (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE

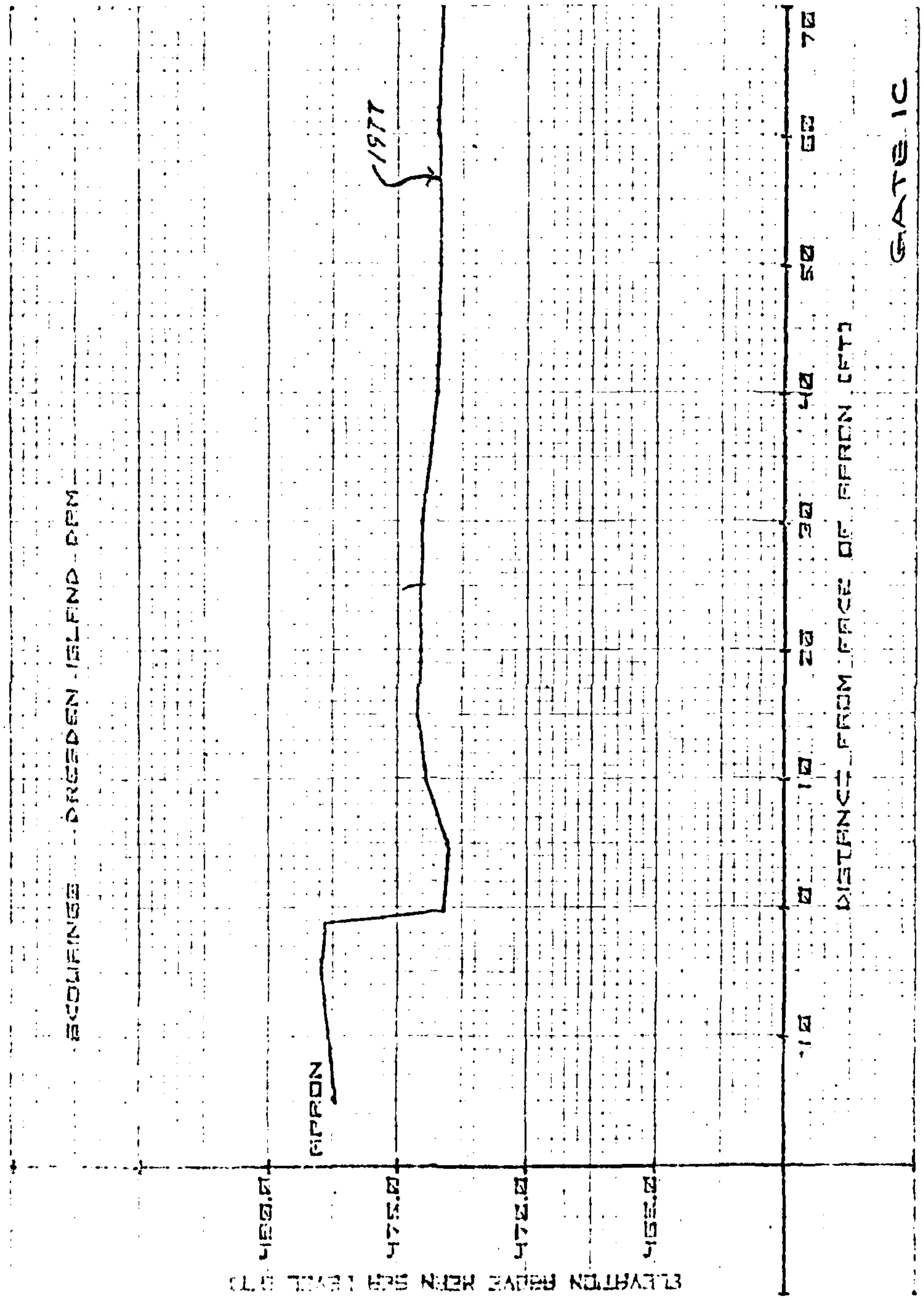
APPENDIX A

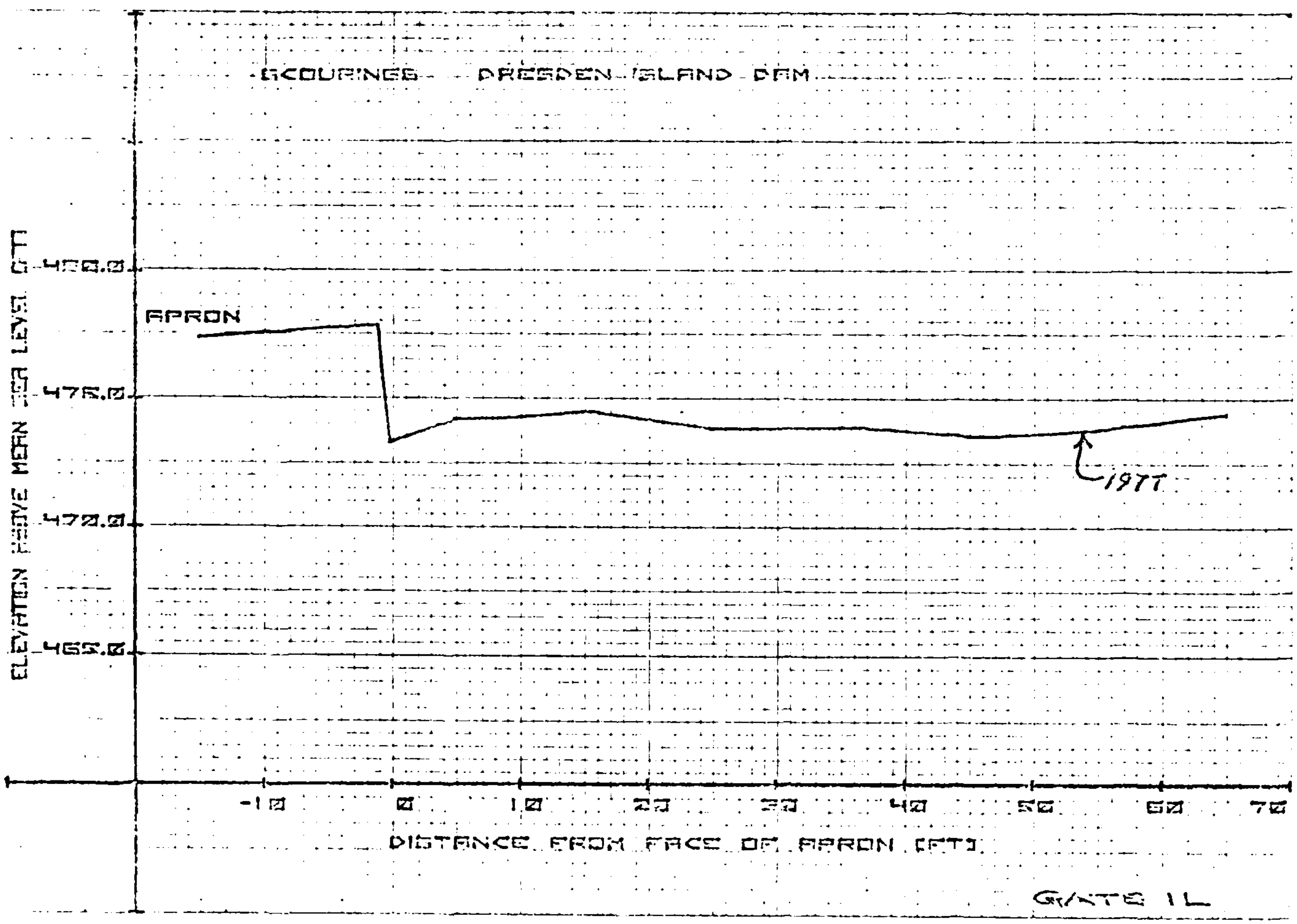
SCOUR PROFILES FROM CHICAGO DISTRICT DRESDEN ISLAND LOCK AND DAM

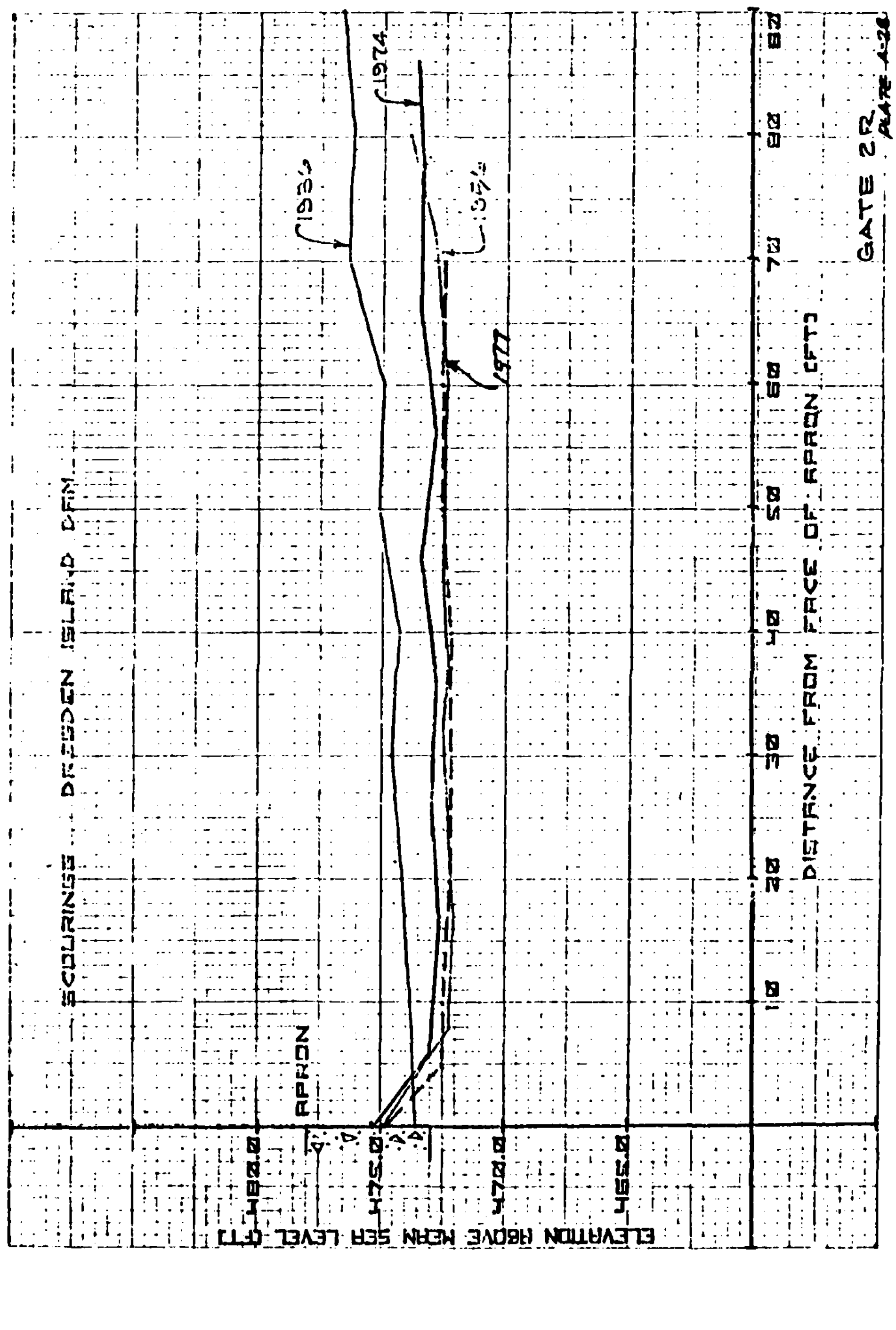

GATES
TOE OF SCOUR PROTECTION
FACE OF APRON
DEPTH OF SCOUR PROTECTION AT THE TOE OF APRON
1977
SCALES: VERT. 1" = 5'
HORZ. 1" = 80'
DRESDEN ISLAND L&D
MAJOR REHABILITATION
SCOUR PROTECTION DAM
PLAN AND PROFILE
AUG. 1977
PLATE 33

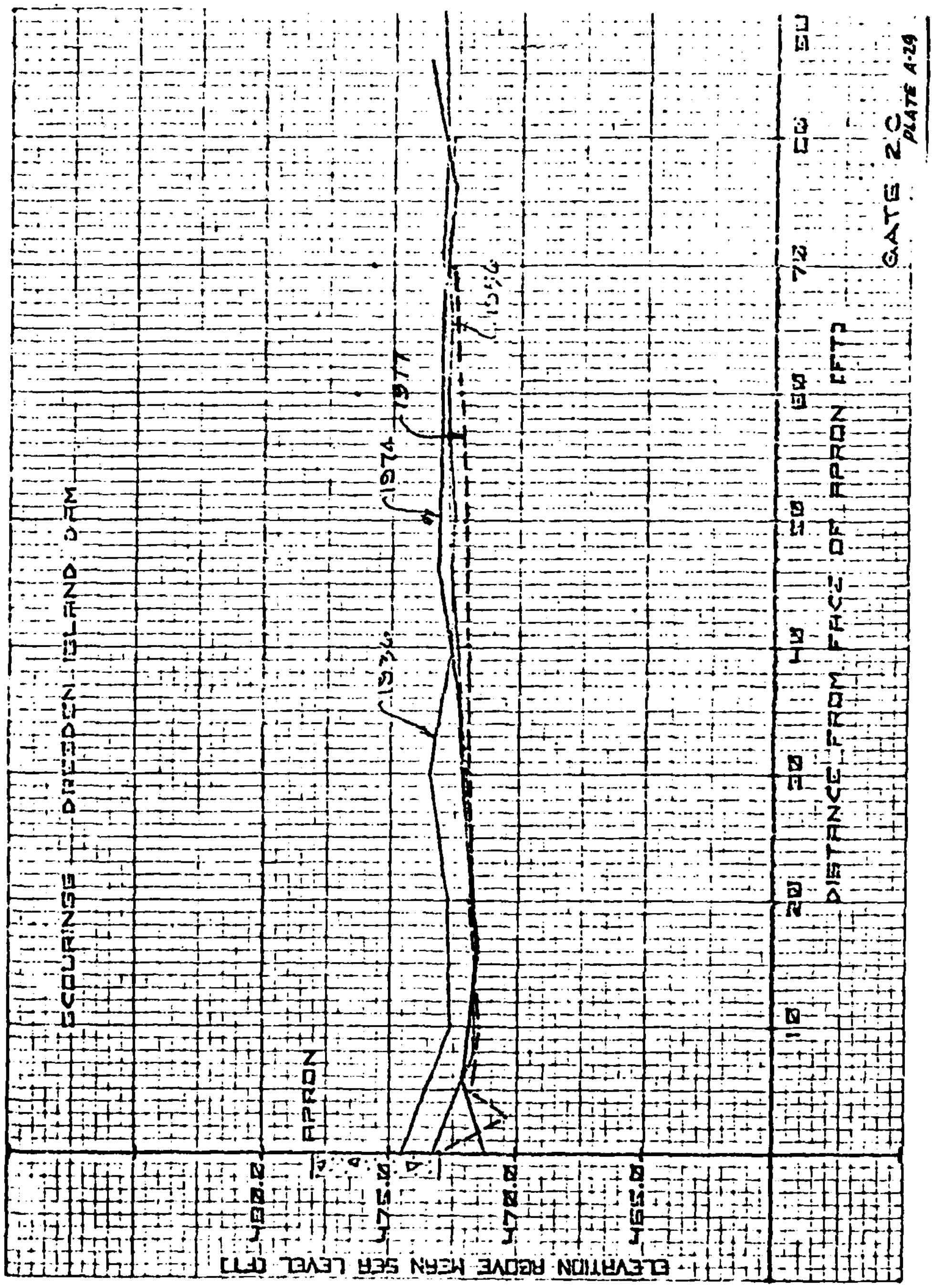

TOP APRON ELEVATION 473.0
BOTTOM APRON ELEVATION 473.0
0'
FACE OF APRON
9R
9C
9L
8R
8C
8L
7R
0'
7C
7L
6R
6C
6L
5R
5C
5L
4R
4C
4L
3R
3C
0'
3L
2R
2C
2L
1R
1C
1L
55'
41'
45'
GATES
BASELINE FOR SCOURINGS (WOOD PLANKING AT BOTTOM OF TAINTOR GATES)
FLOW
UPPER FLAT POOL ELEVATION 505.0
3' WIDE LEDGE AT FACE OF APRON & LOCATED BELOW TOP OF APRON
SCALE: 1 INCH = 100 FEET
DRESDEN ISLAND DAM
ILLINOIS WATERWAY
SCOURING STATIONS
PLATE A-26

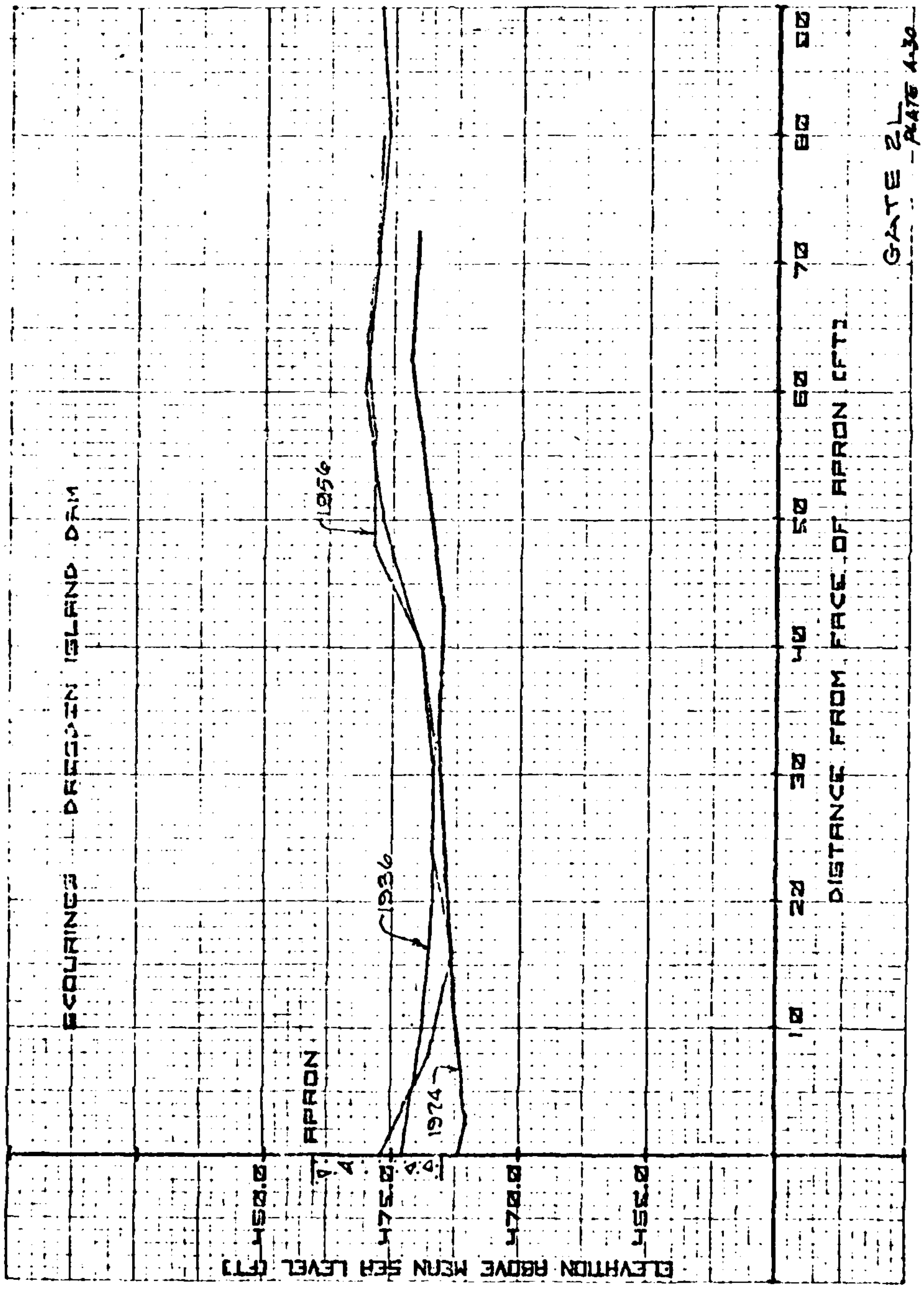

SCOURINGS — DRESDEN ISLAND DAM
APRON
1936
1974
1977
ELEVATION ABOVE MEAN SEA LEVEL (FT)
480.0
475.0
470.0
465.0
10
20
30
40
50
60
70
80
90
DISTANCE FROM FACE OF APRON (FT)
GATE 1R
PLATE A-27

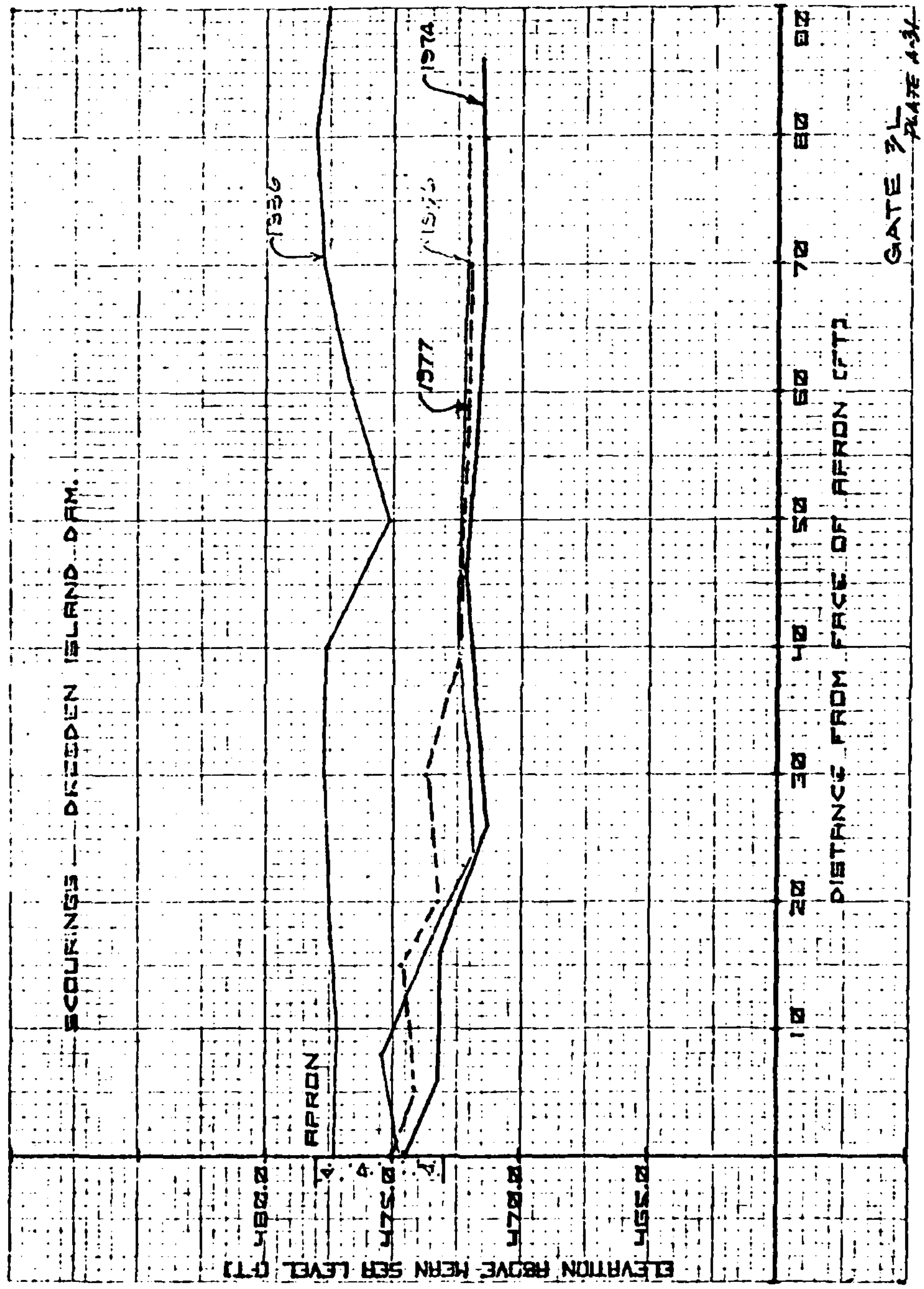

SCOURINGS — DRESDEN ISLAND DAM
GATE 1C
1977
APRON
DISTANCE FROM FACE OF APRON (FT)
-10
0
10
20
30
40
50
60
70
ELEVATION ABOVE MEAN SEA LEVEL (FT)
480.0
475.0
470.0
465.0


SCOURINGS — DRESDEN ISLAND DAM
ELEVATION ABOVE MEAN SEA LEVEL (FT)
480.0
475.0
470.0
465.0
APRON
1977
-10
0
10
20
30
40
50
60
70
DISTANCE FROM FACE OF APRON (FT)
GATE 1L

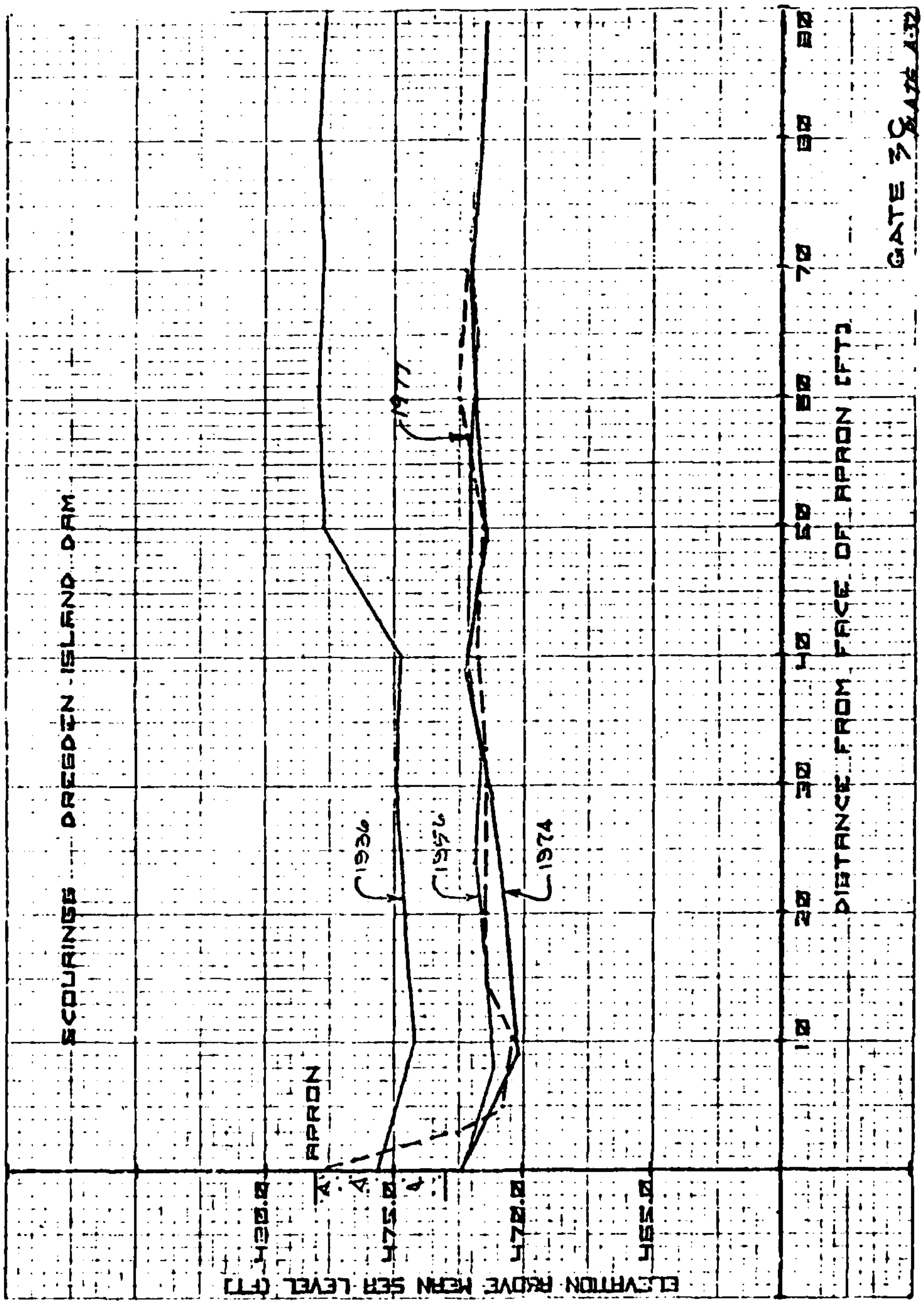

SCOURINGS DRESDEN ISLAND DAM
ELEVATION ABOVE MEAN SEA LEVEL (FT)
DISTANCE FROM FACE OF APRON (FT)
APRON
480.0
475.0
470.0
465.0
10
20
30
40
50
60
70
1974
1977
GATE 2R


SCOURINGS — DRESDEN ISLAND DAM
ELEVATION ABOVE MEAN SEA LEVEL (FT)
480.0
475.0
470.0
465.0
APRON
DISTANCE FROM FACE OF APRON (FT)
10
20
30
40
50
60
70
80
90
1976
1974
1977
1976
GATE 2C
PLATE A-29

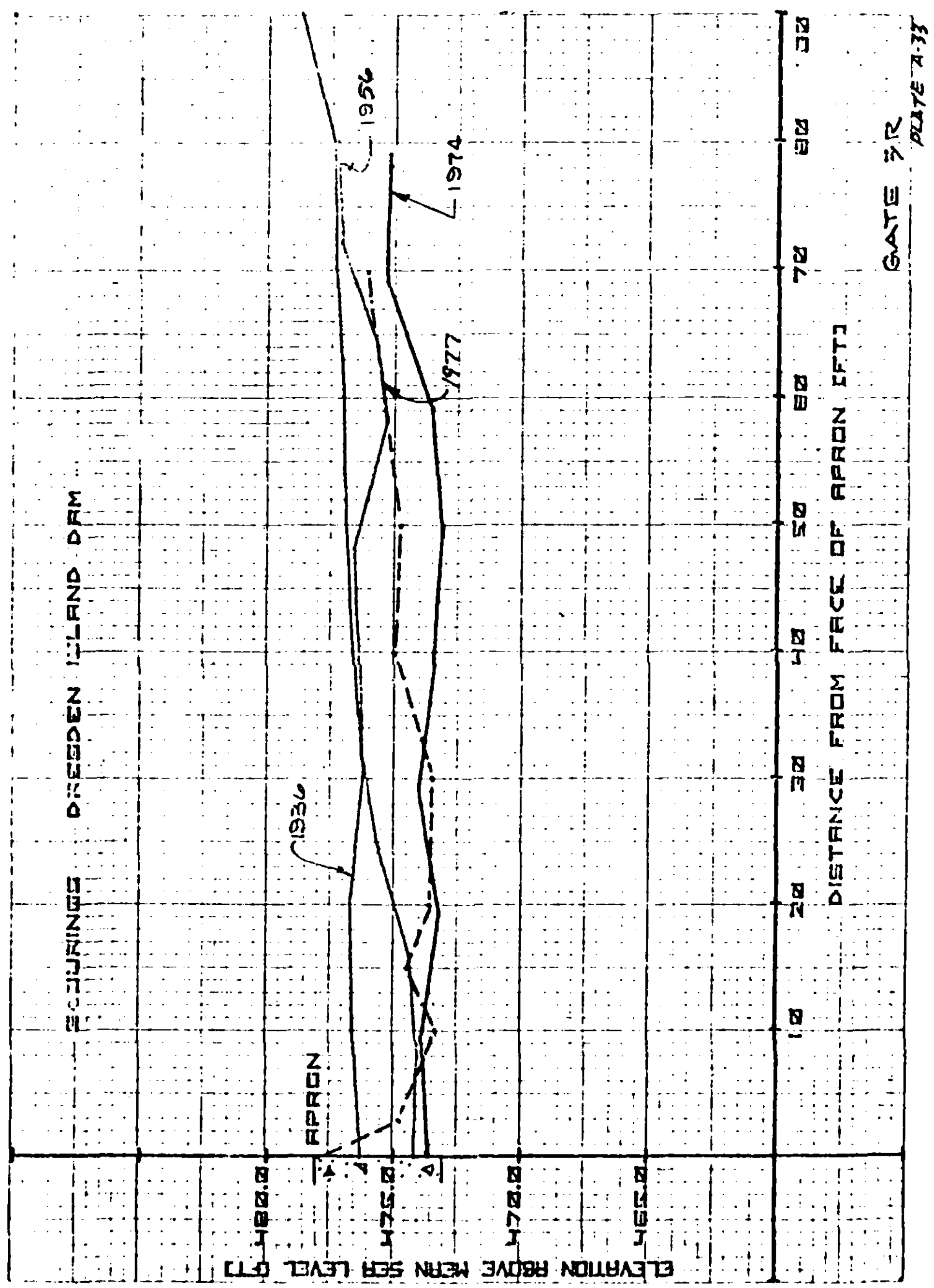

SCOURING DRESDEN ISLAND DAM
GATE 2L
PLATE A-30
DISTANCE FROM FACE OF APRON (FT)
ELEVATION ABOVE MEAN SEA LEVEL (FT)
APRON
1956
1936
1924
10
20
30
40
50
60
70
80
90
480.0
475.0
470.0
465.0

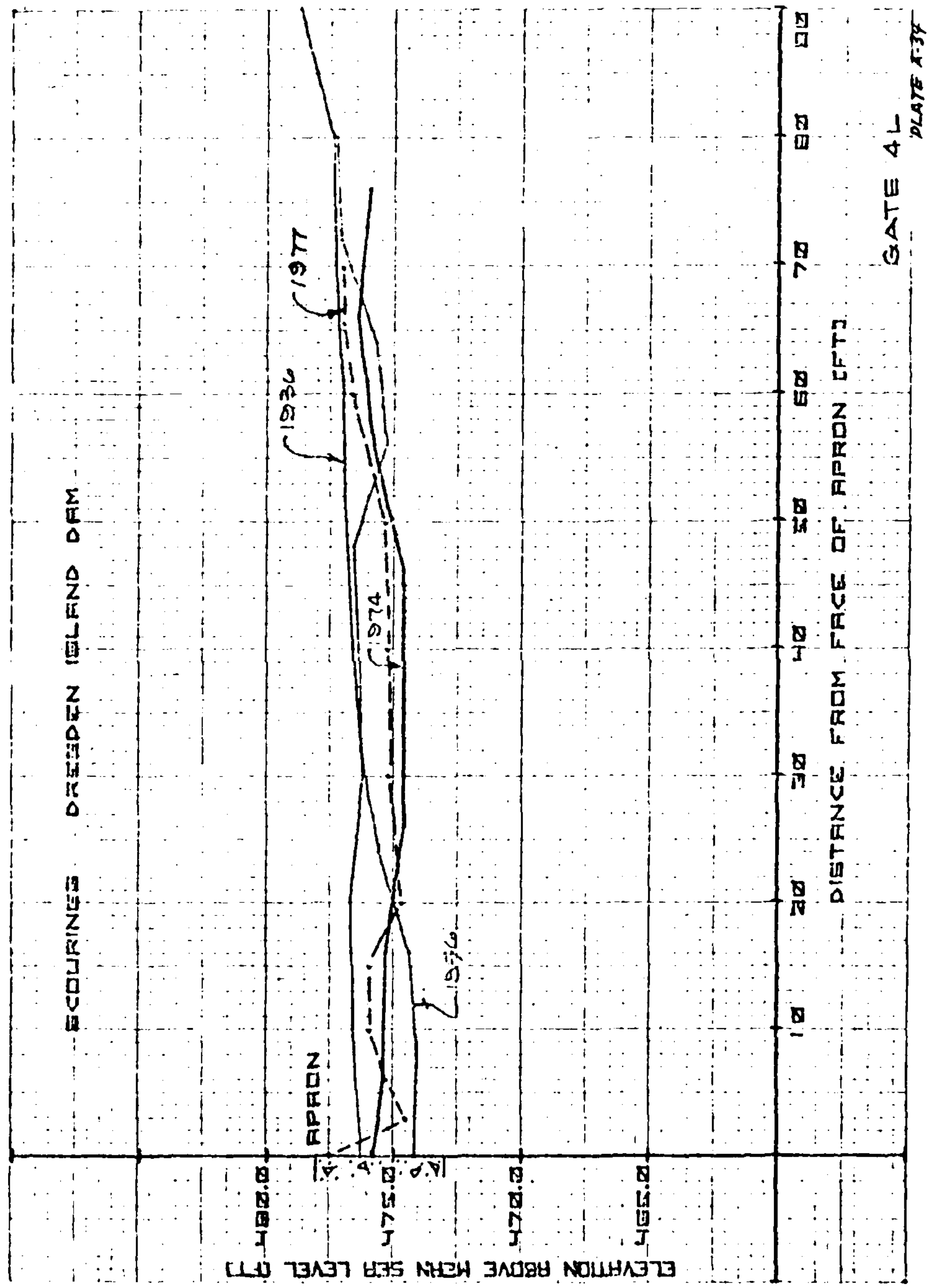


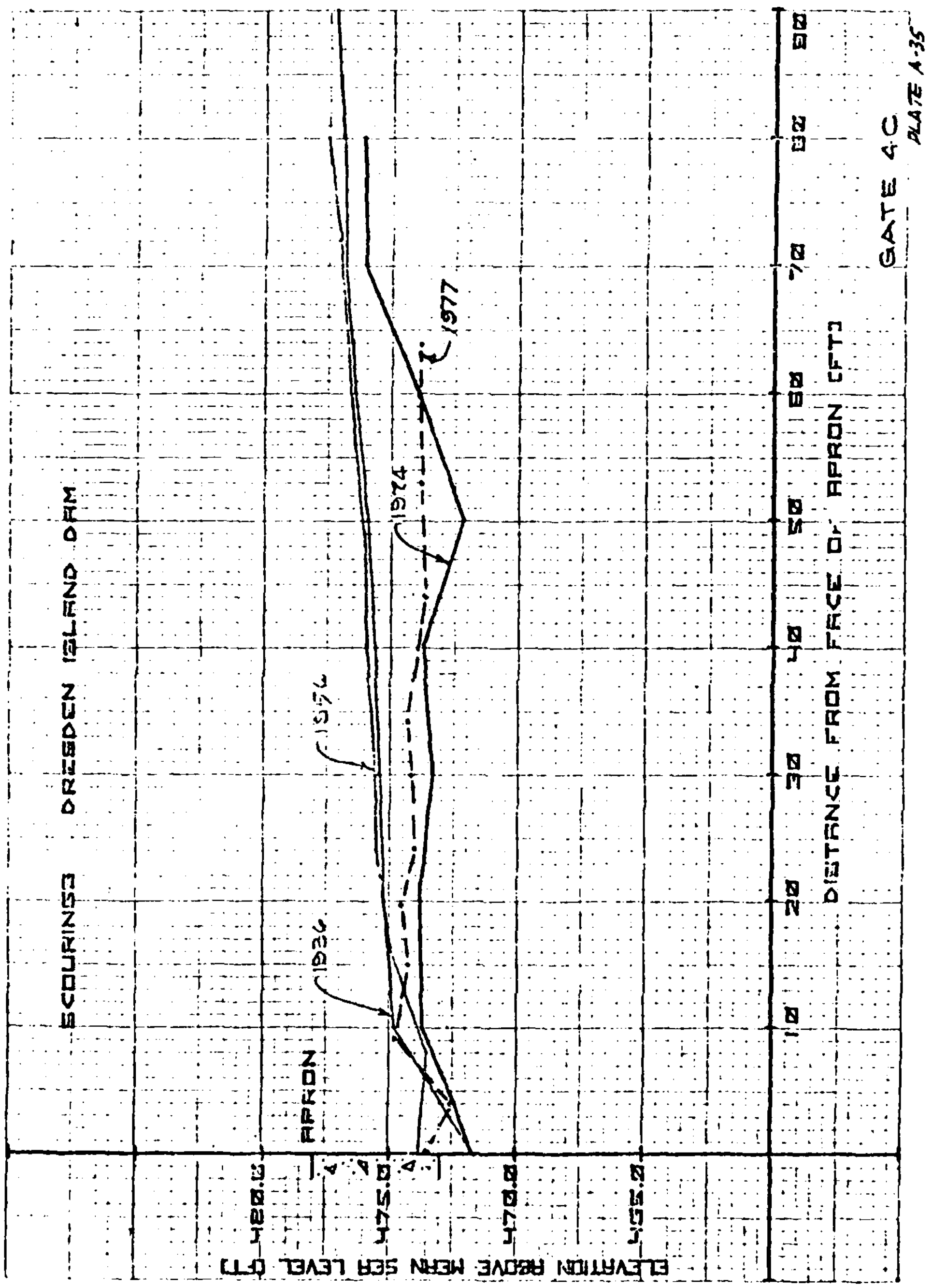
SCOURINGS — DRESDEN ISLAND DAM.
APRON
ELEVATION ABOVE MEAN SEA LEVEL (FT)
480.0
475.0
470.0
465.0
DISTANCE FROM FACE OF APRON (FT)
10
20
30
40
50
60
70
80
90
1936
1977
1974
GATE 7L

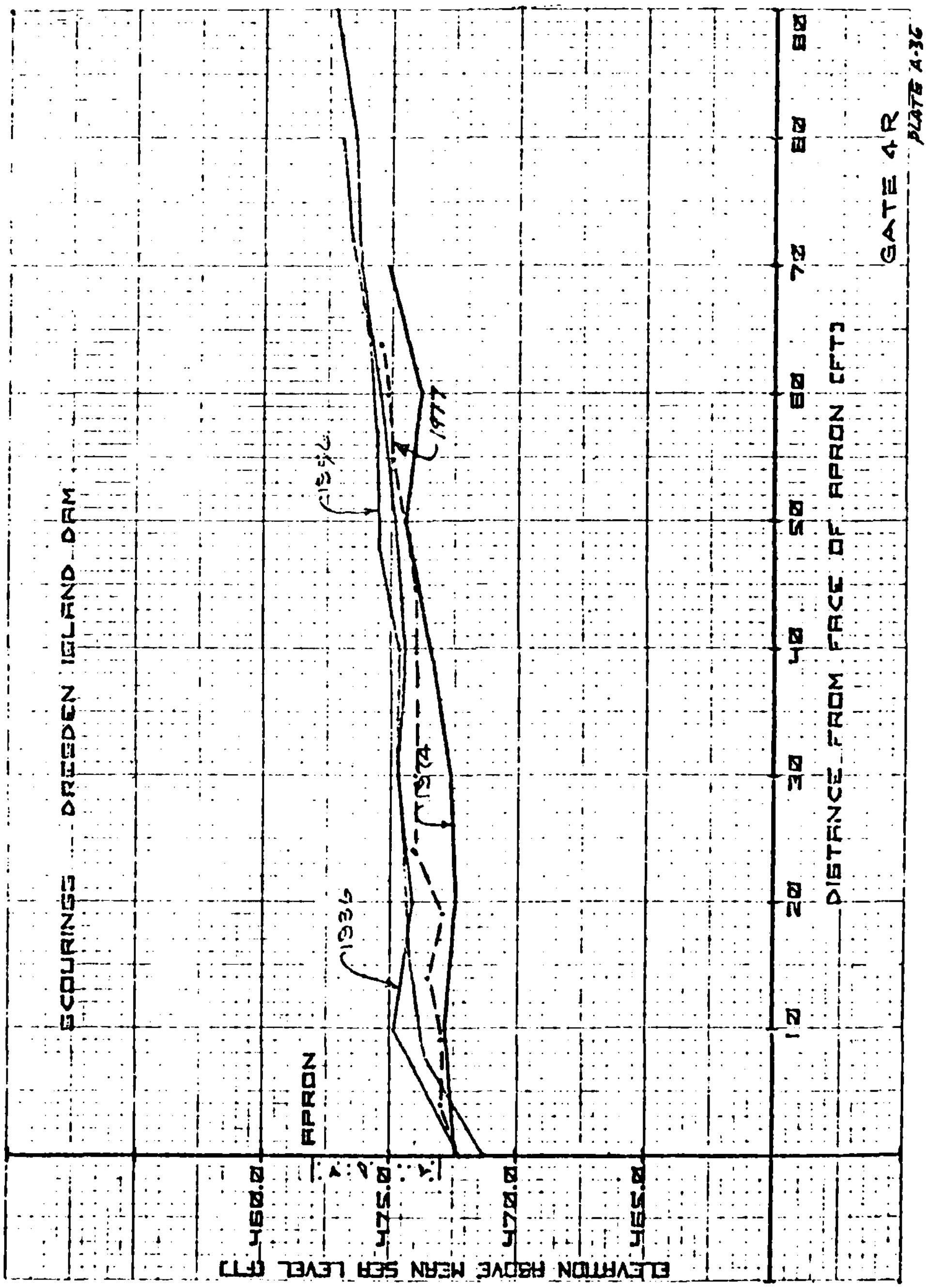


SCOURINGS DRESDEN ISLAND DAM
GATE 3C
DISTANCE FROM FACE OF APRON (FT)
ELEVATION ABOVE MEAN SEA LEVEL (FT)
APRON
1936
1956
1974
1977
10
20
30
40
50
60
70
80
90
480.0
475.0
470.0
465.0

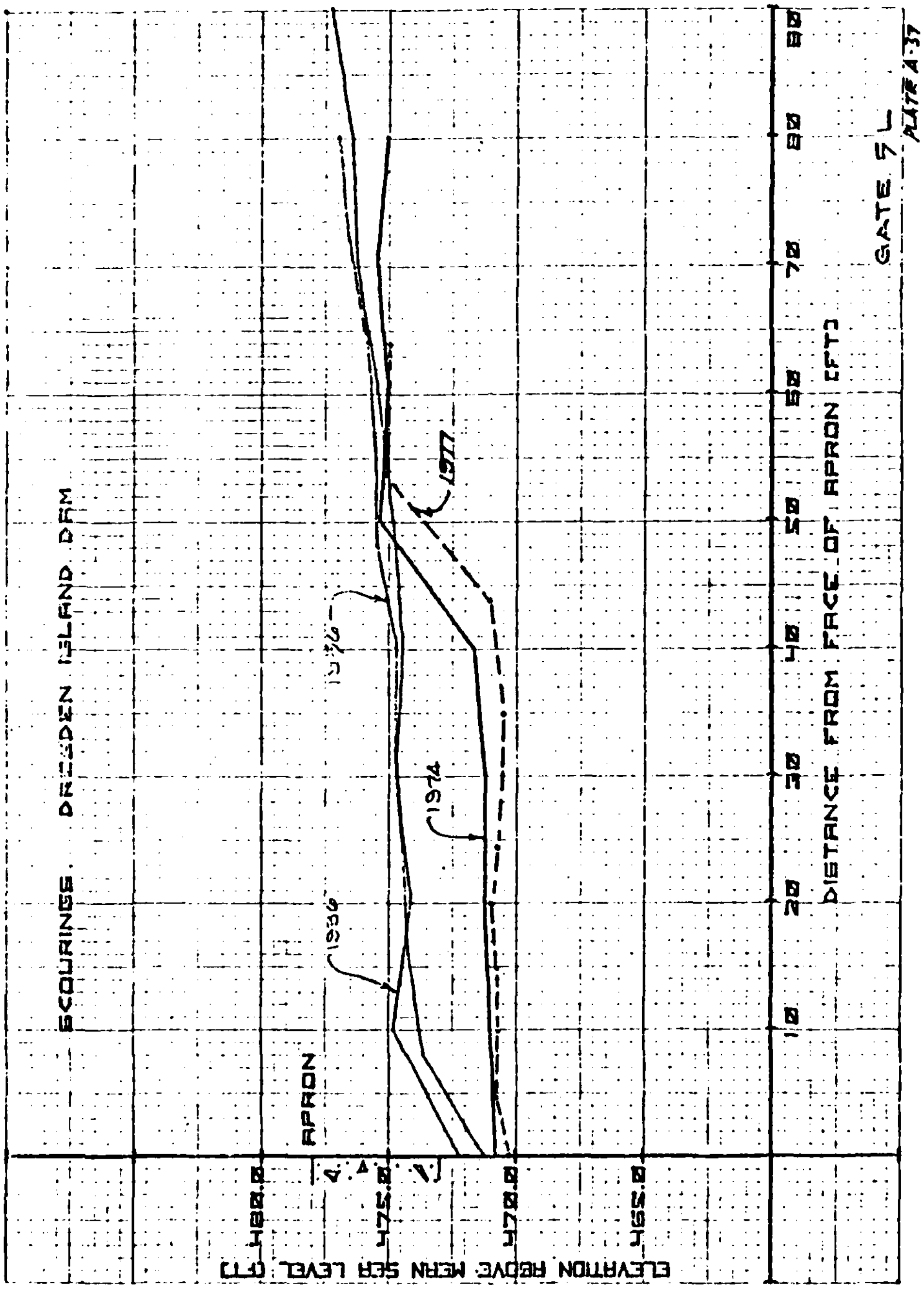

SCOURINGS DRESDEN ISLAND DAM
ELEVATION ABOVE MEAN SEA LEVEL (FT)
480.0
475.0
470.0
465.0
APRON
DISTANCE FROM FACE OF APRON (FT)
10
20
30
40
50
60
70
80
90
1936
1956
1974
1977
GATE 7R
PLATE A-33

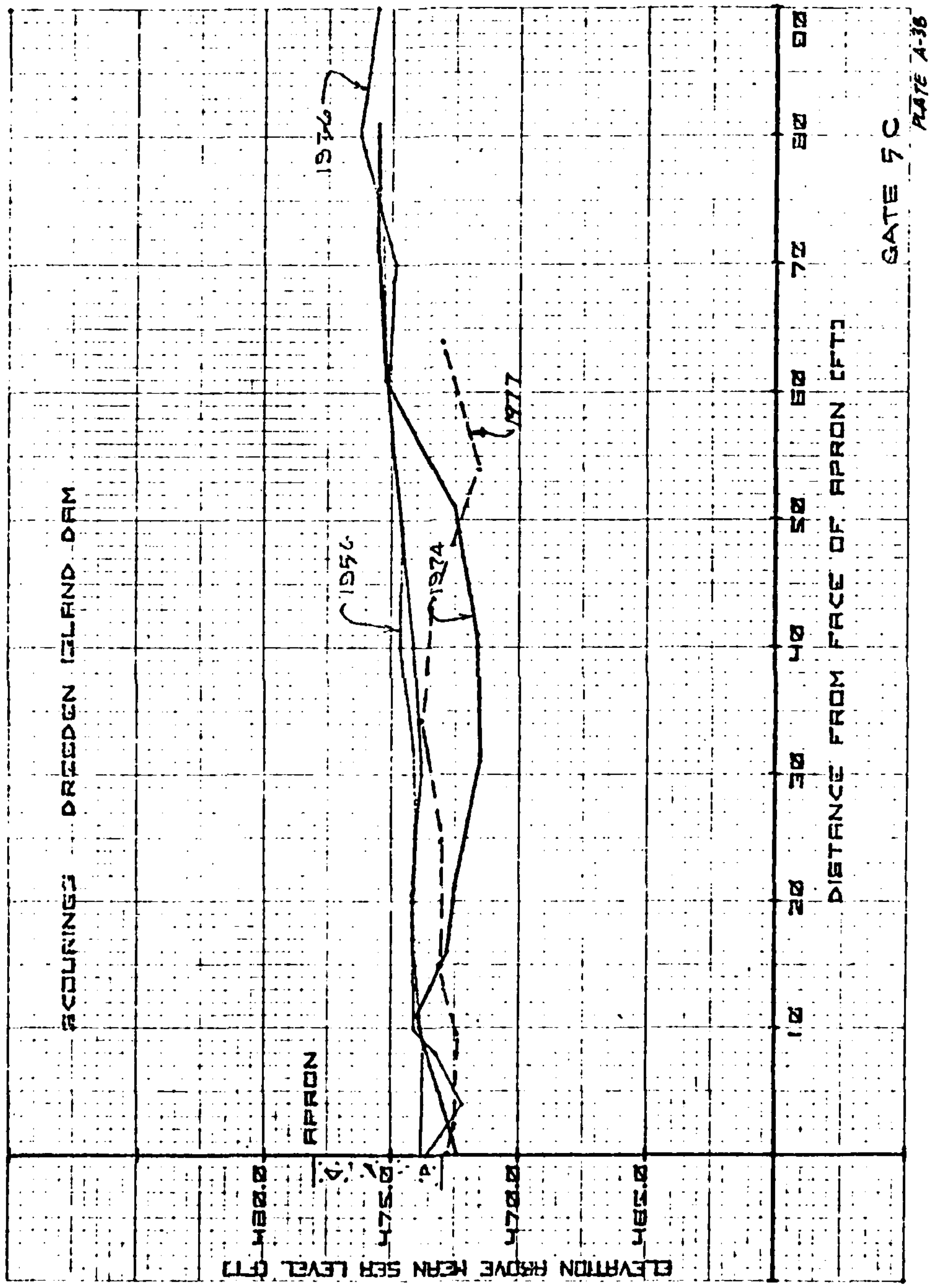

SCOURINGS DRESDEN ISLAND DAM
APRON
1977
1936
1974
1956
480.0
475.0
470.0
465.0
ELEVATION ABOVE MEAN SEA LEVEL (FT)
10
20
30
40
50
60
70
80
90
DISTANCE FROM FACE OF APRON (FT)
GATE 4L
PLATE X-34


SCOURINGS DRESDEN ISLAND DAM
GATE 4C
PLATE A-35
APRON
1936
1956
1974
1977
480.0
475.0
470.0
465.0
ELEVATION ABOVE MEAN SEA LEVEL (FT)
10
20
30
40
50
60
70
80
90
DISTANCE FROM FACE OF APRON (FT)



SCOURINGS — DRESDEN ISLAND DAM
GATE 4R
PLATE A-36
DISTANCE FROM FACE OF APRON (FT)
ELEVATION ABOVE MEAN SEA LEVEL (FT)
APRON
1936
1956
1974
1977
480.0
475.0
470.0
465.0
10
20
30
40
50
60
70
80
90


SCOURINGS. DRESDEN ISLAND DAM
APRON
1936
1974
1976
1977
480.0
475.0
470.0
465.0
ELEVATION ABOVE MEAN SEA LEVEL (FT)
10
20
30
40
50
60
70
80
90
DISTANCE FROM FACE OF APRON (FT)
GATE 7L
PLATE A-37


SCOURINGS DRESDEN ISLAND DAM
APRON
1956
1974
1976
1977
480.0
475.0
470.0
465.0
ELEVATION ABOVE MEAN SEA LEVEL (FT)
10
20
30
40
50
60
70
80
90
DISTANCE FROM FACE OF APRON (FT)
GATE 5 C
PLATE A-38

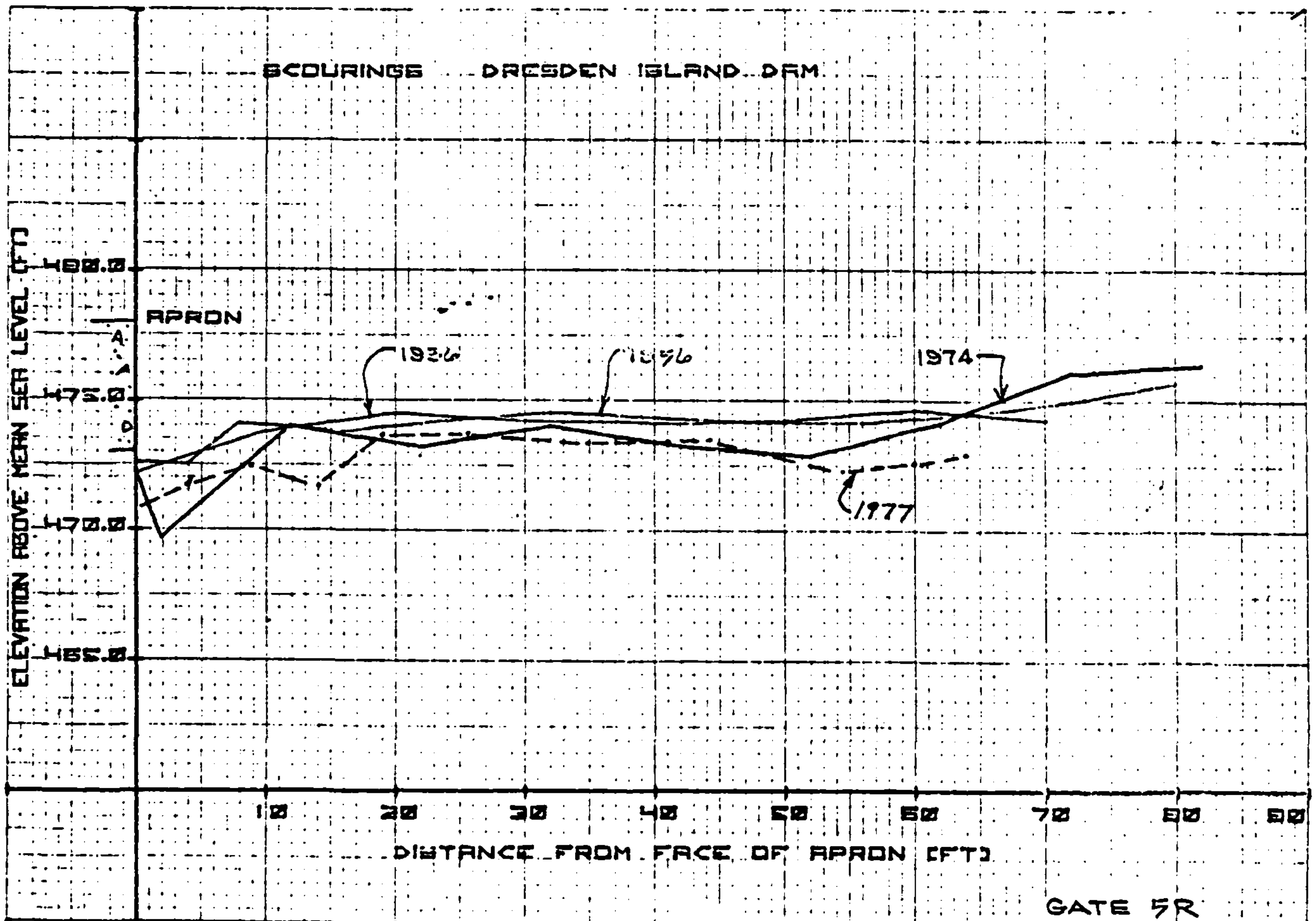
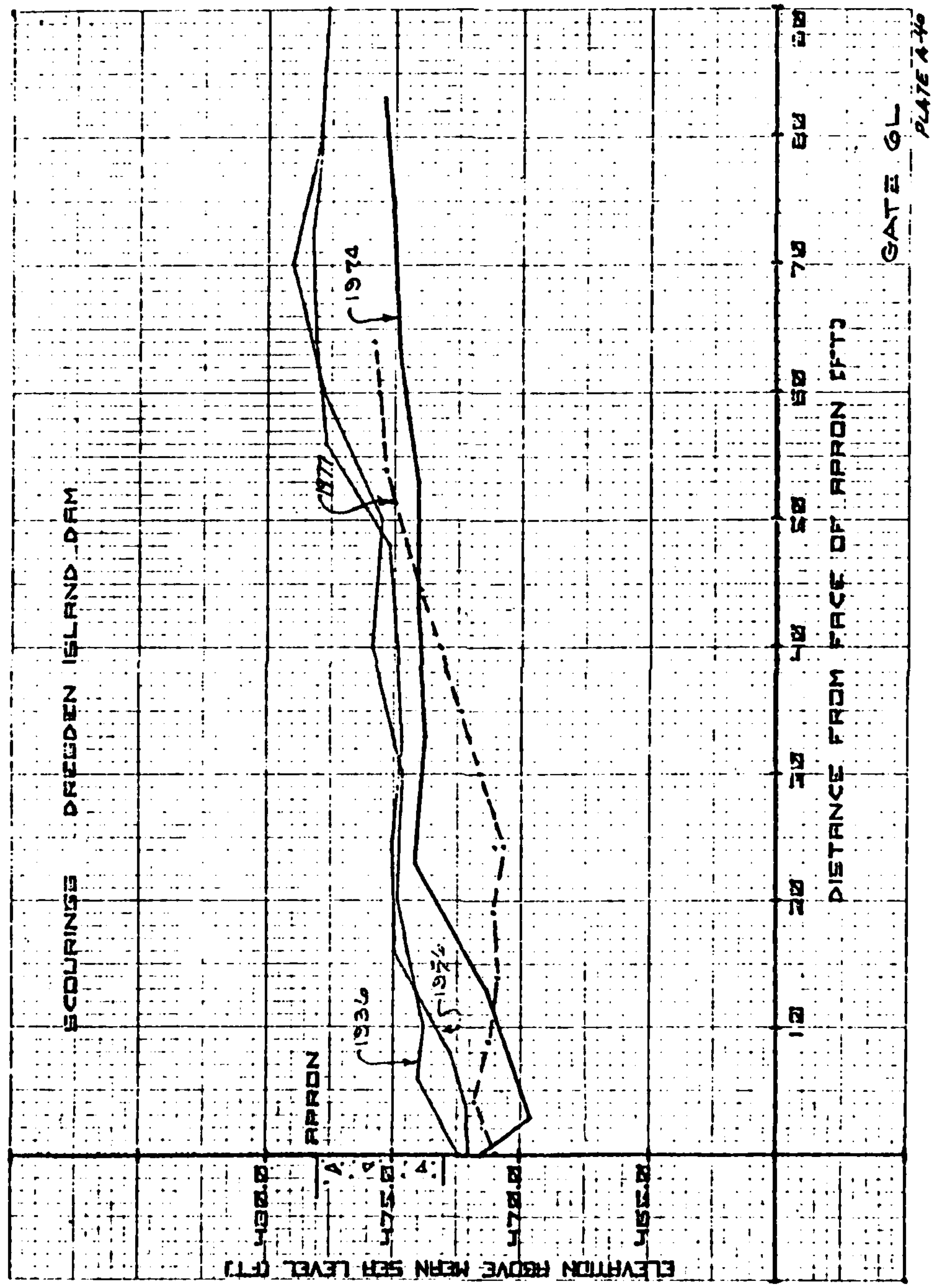
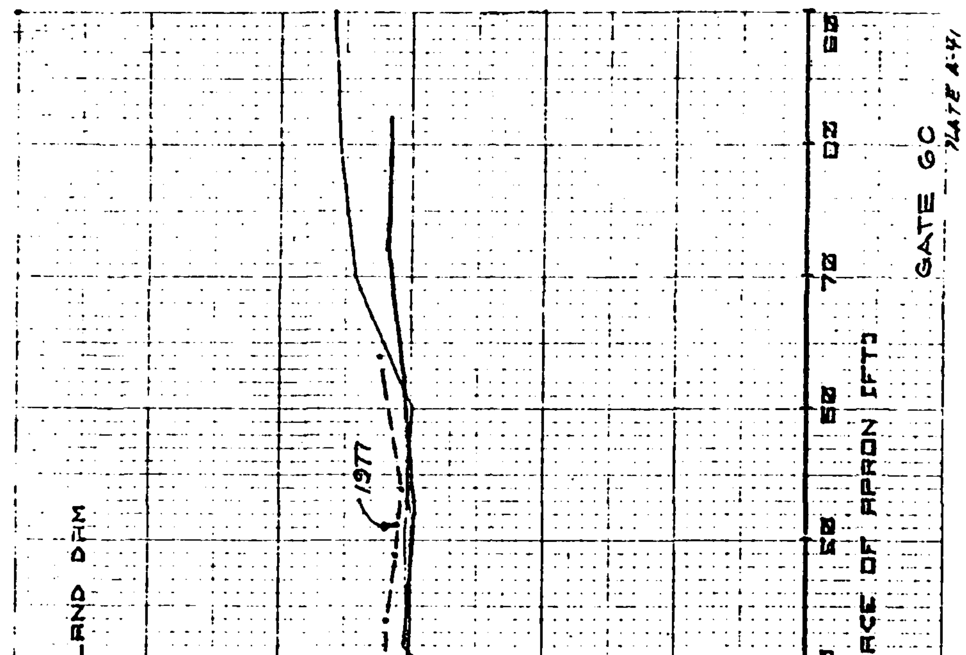
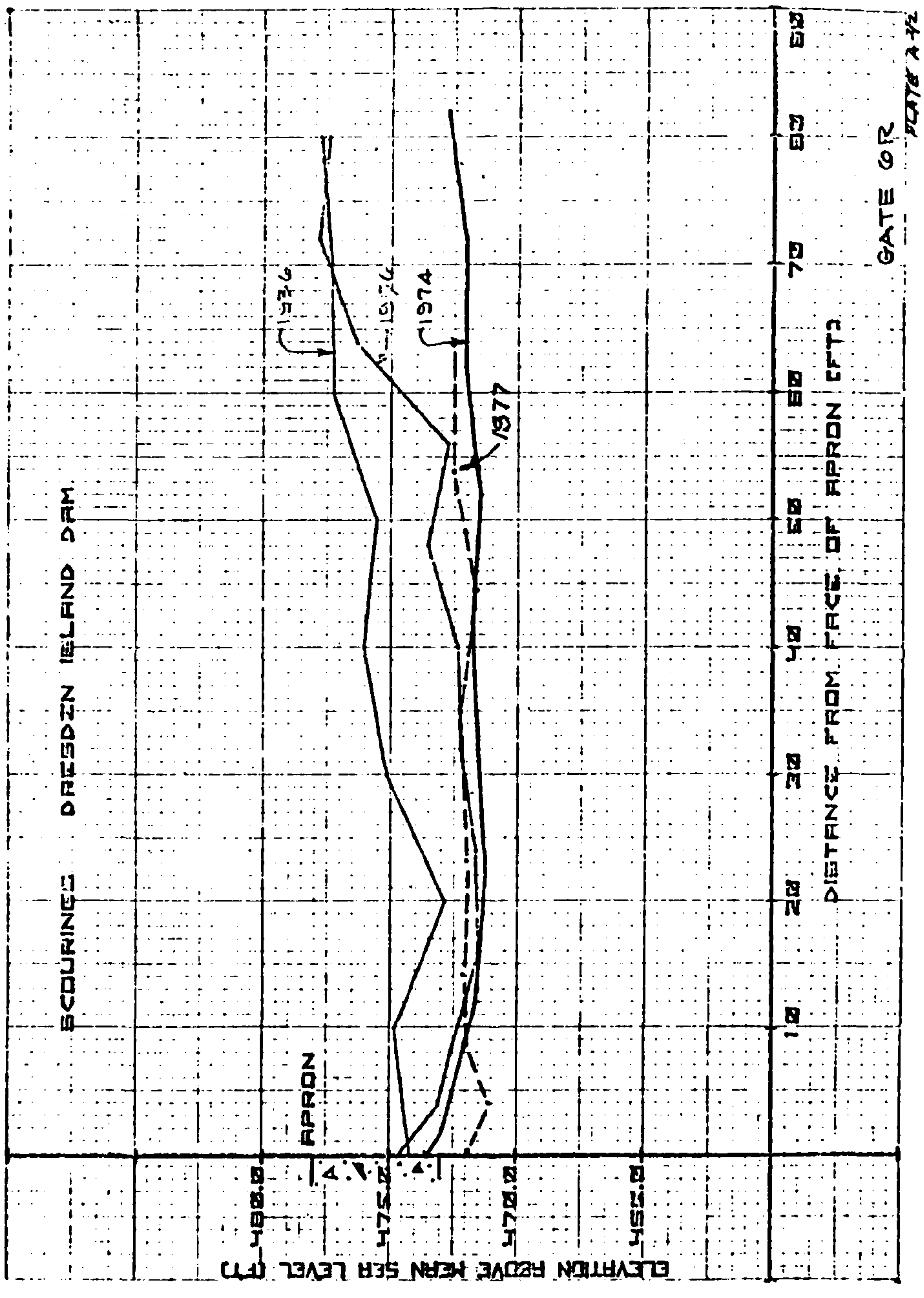

SCOURINGS DRESDEN ISLAND DAM
ELEVATION ABOVE MEAN SEA LEVEL (FT)
480.0
475.0
470.0
465.0
APRON
1936
1956
1974
1977
10
20
30
40
50
60
70
80
90
DISTANCE FROM FACE OF APRON (FT)
GATE 5R

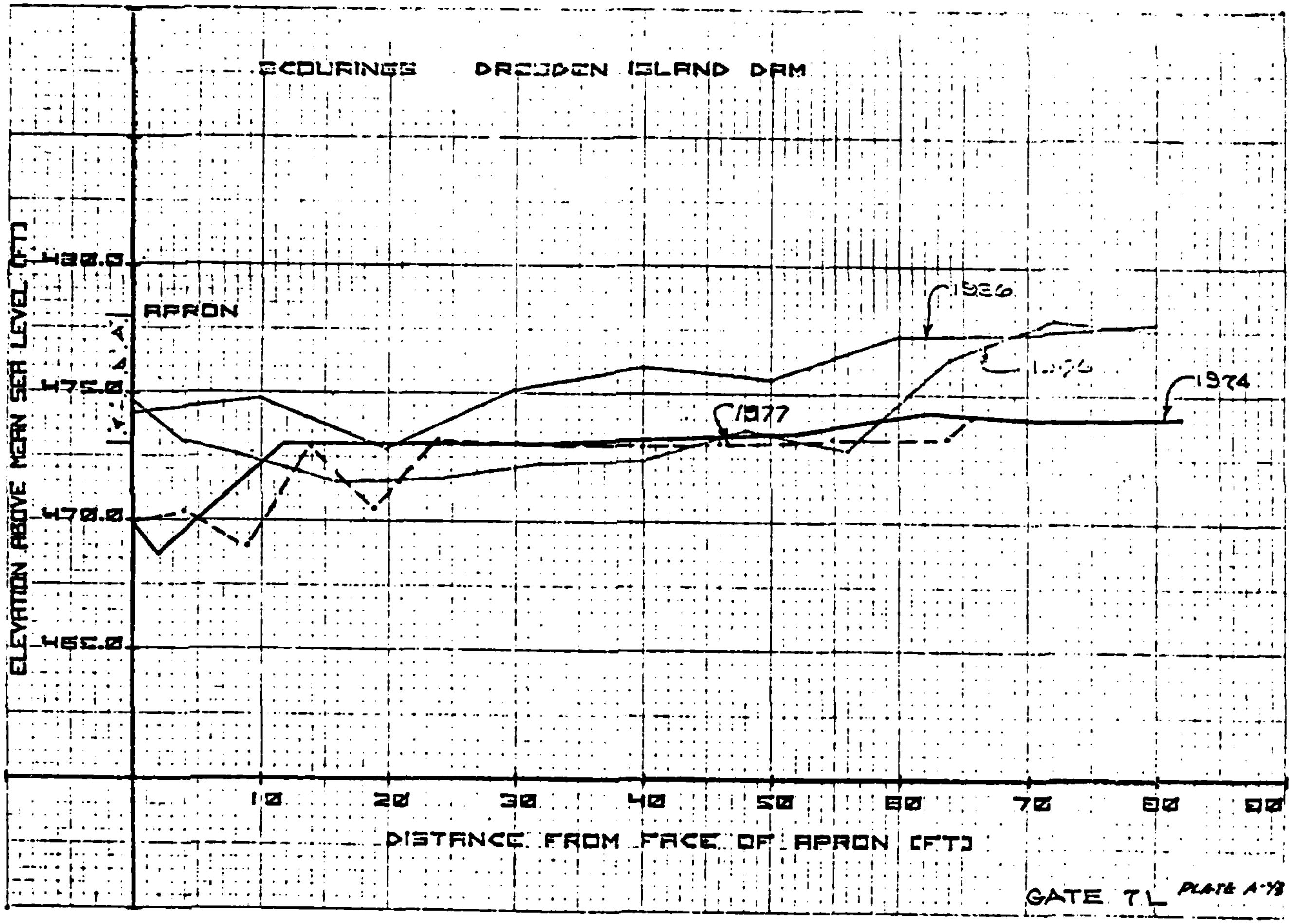
PLATE A-39

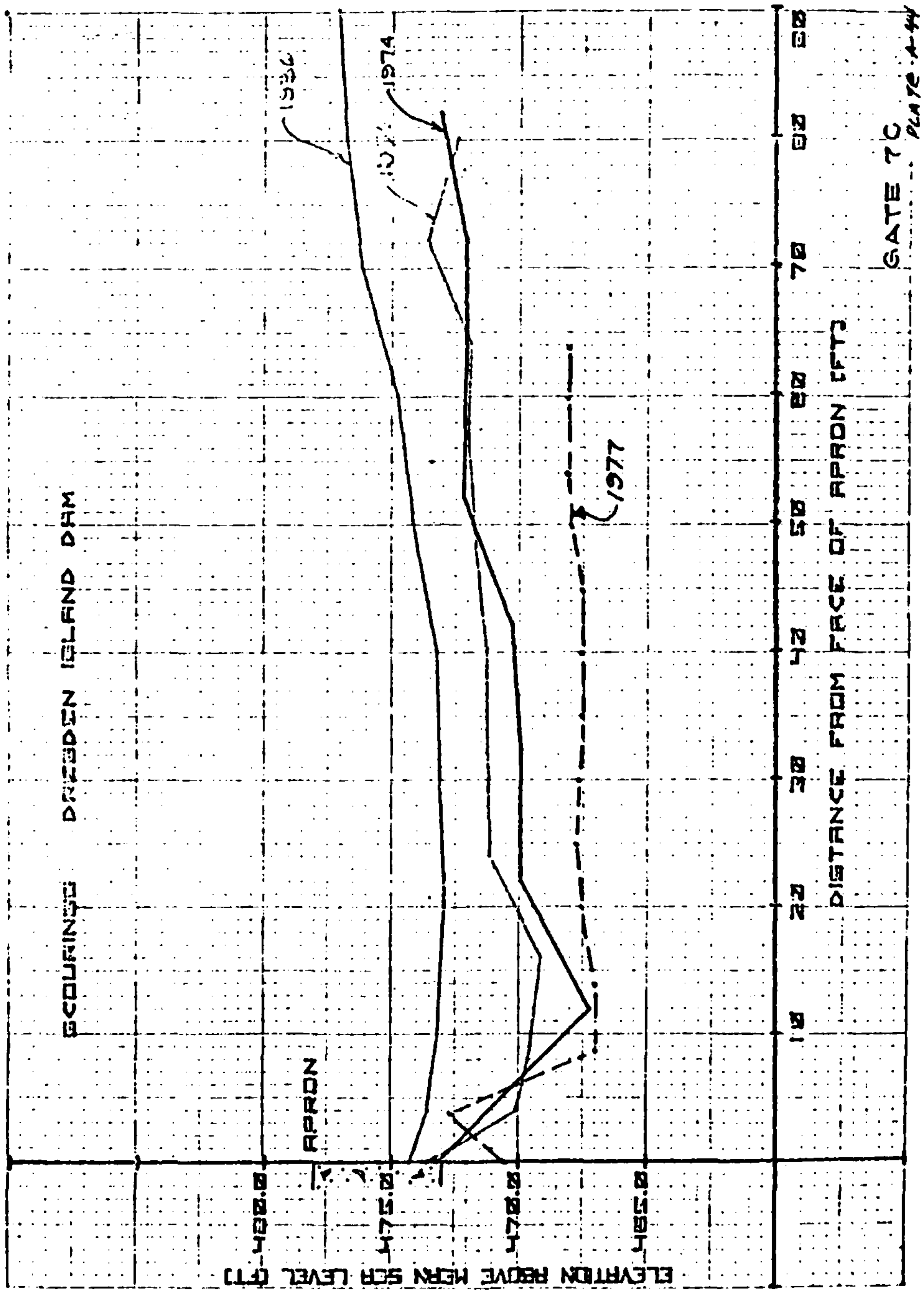


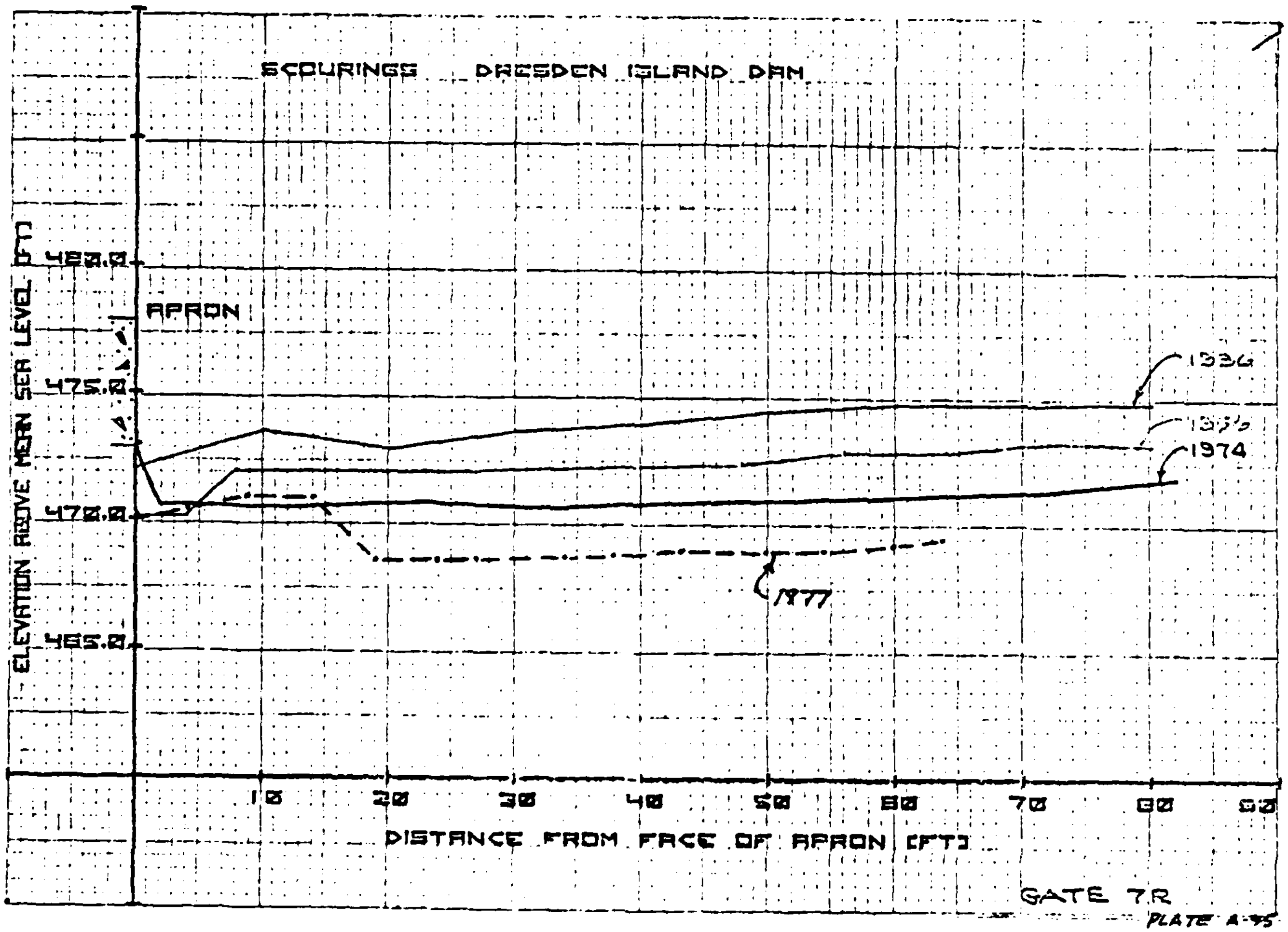
SCOURINGS DRESDEN ISLAND DAM
DISTANCE FROM FACE OF APRON (FT)
ELEVATION ABOVE MEAN SEA LEVEL (FT)
APRON
1974
1977
1976
1936
GATE 6L
PLATE A-46

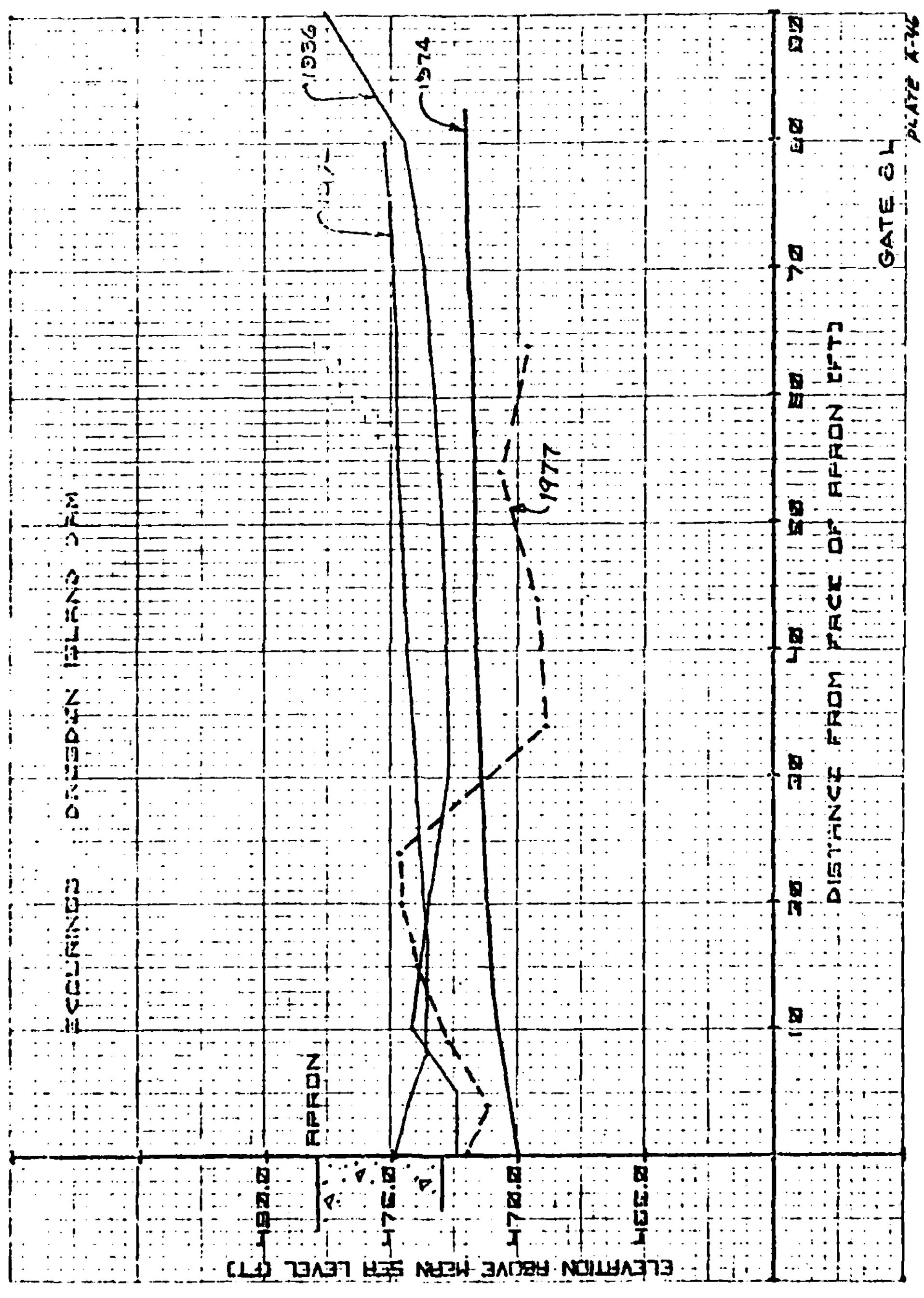

SCOURINGS DRESDEN ISLAND DAM
1977
1936
1956
1974
APRON
480.0
475.0
470.0
465.0
ELEVATION ABOVE MEAN SEA LEVEL (FT)
10
20
30
40
50
60
70
80
90
DISTANCE FROM FACE OF APRON (FT)
GATE 6C
PLATE A-41

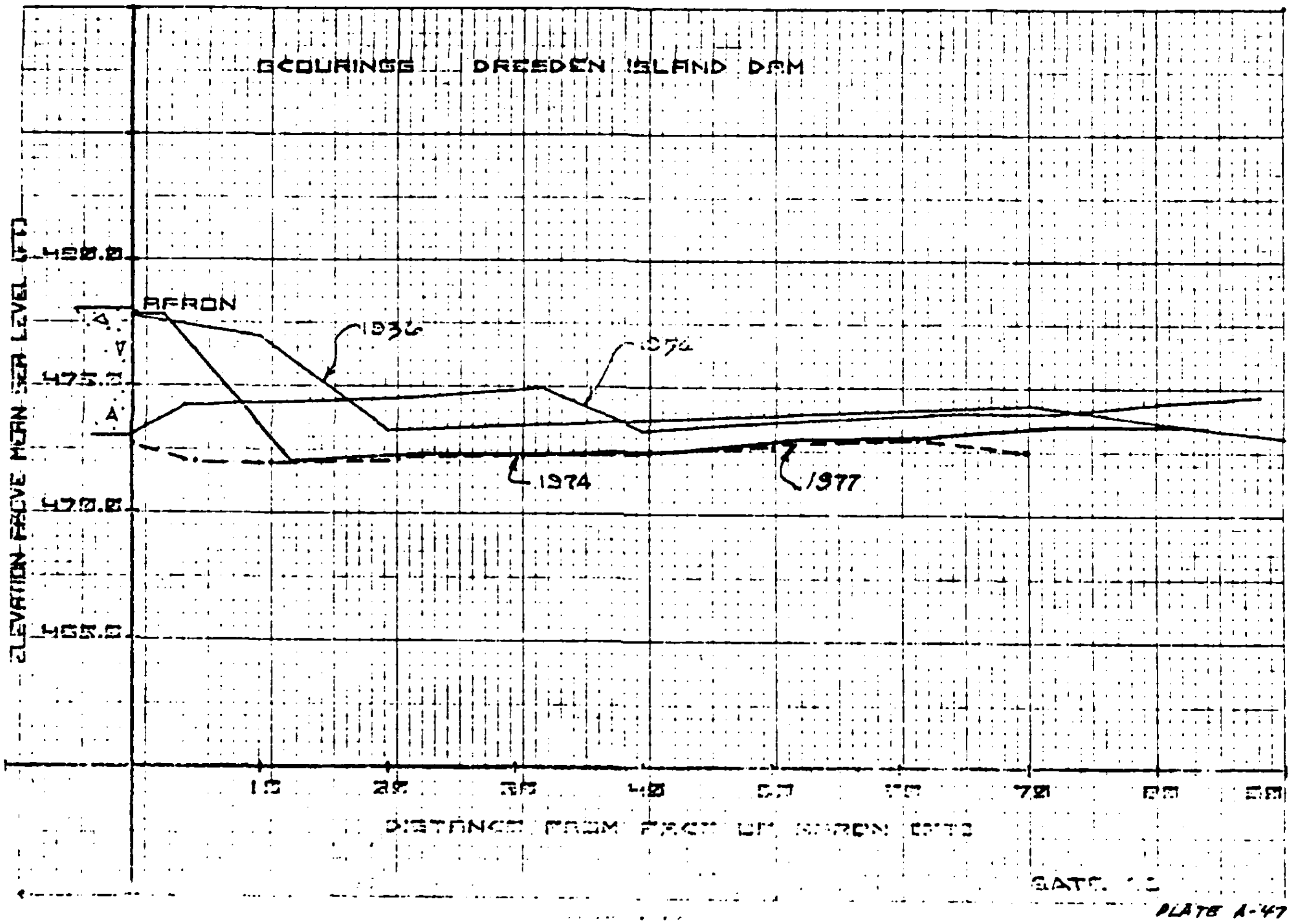

SCOURING DRESDEN ISLAND DAM
GATE 6R
APRON
1976
1974
1977
10
20
30
40
50
60
70
80
90
480.0
475.0
470.0
465.0
DISTANCE FROM FACE OF APRON (FT)
ELEVATION ABOVE MEAN SEA LEVEL (FT)


SCOURINGS DRESDEN ISLAND DAM
ELEVATION ABOVE MEAN SEA LEVEL (FT)
480.0
475.0
470.0
465.0
APRON
1936
1974
1977
10
20
30
40
50
60
70
80
90
DISTANCE FROM FACE OF APRON (FT)
GATE 7L
PLATE A-73

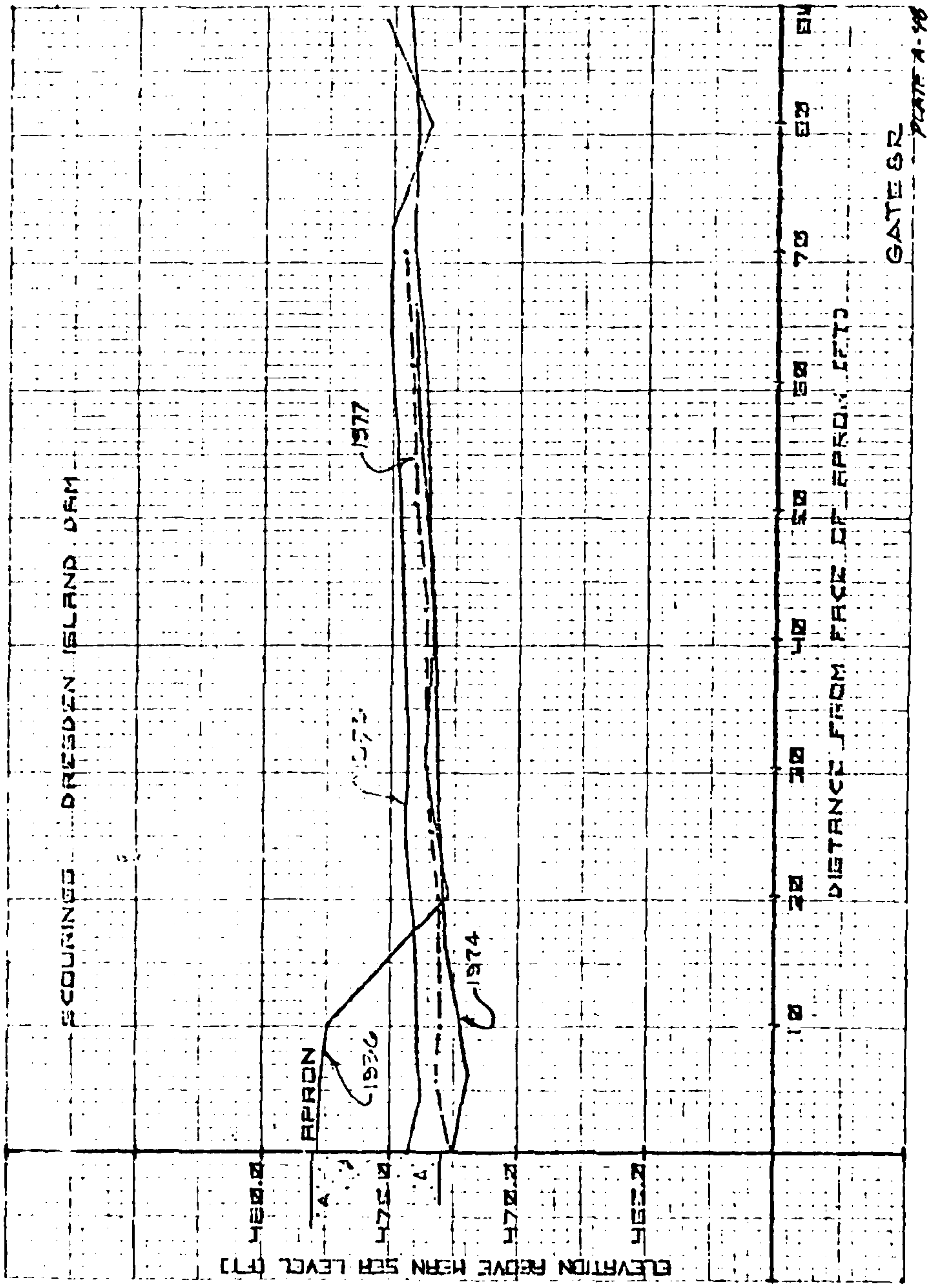

SCOURING DRESDEN ISLAND DAM
APRON
1986
1974
1977
480.0
475.0
470.0
465.0
ELEVATION ABOVE MEAN SEA LEVEL (FT)
10
20
30
40
50
60
70
80
90
DISTANCE FROM FACE OF APRON (FT)
GATE 7C
PLATE A-44


SCOURINGS DRESDEN ISLAND DAM
ELEVATION ABOVE MEAN SEA LEVEL (FT)
480.0
475.0
470.0
465.0
APRON
1936
1974
1977
10
20
30
40
50
60
70
80
90
DISTANCE FROM FACE OF APRON (FT)
GATE 7R
PLATE A-95


DRESDEN ISLAND DAM
APRON
ELEVATION ABOVE MEAN SEA LEVEL (FT)
482.0
476.0
470.0
1936
1974
1977
10
20
30
40
50
60
70
80
DISTANCE FROM FACE OF APRON (FT)



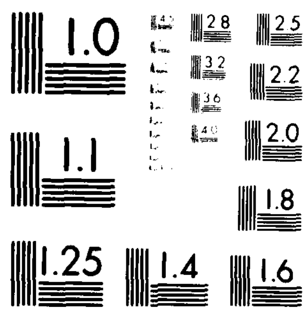
SCOURINGS DRESDEN ISLAND DAM
APRON
1936
1974
1974
1977
480.0
475.0
470.0
465.0
ELEVATION ABOVE MEAN SEA LEVEL (FT)
10
20
30
40
70
PLATE A-47


SCOURINGS DRESDEN ISLAND DAM
DISTANCE FROM FACE OF APRON (FT)
ELEVATION ABOVE MEAN SEA LEVEL (FT)
480.0
475.0
470.0
465.0
10
20
30
40
50
60
70
80
APRON
1974
1976
1977
GATE 5R
PLATE A-98

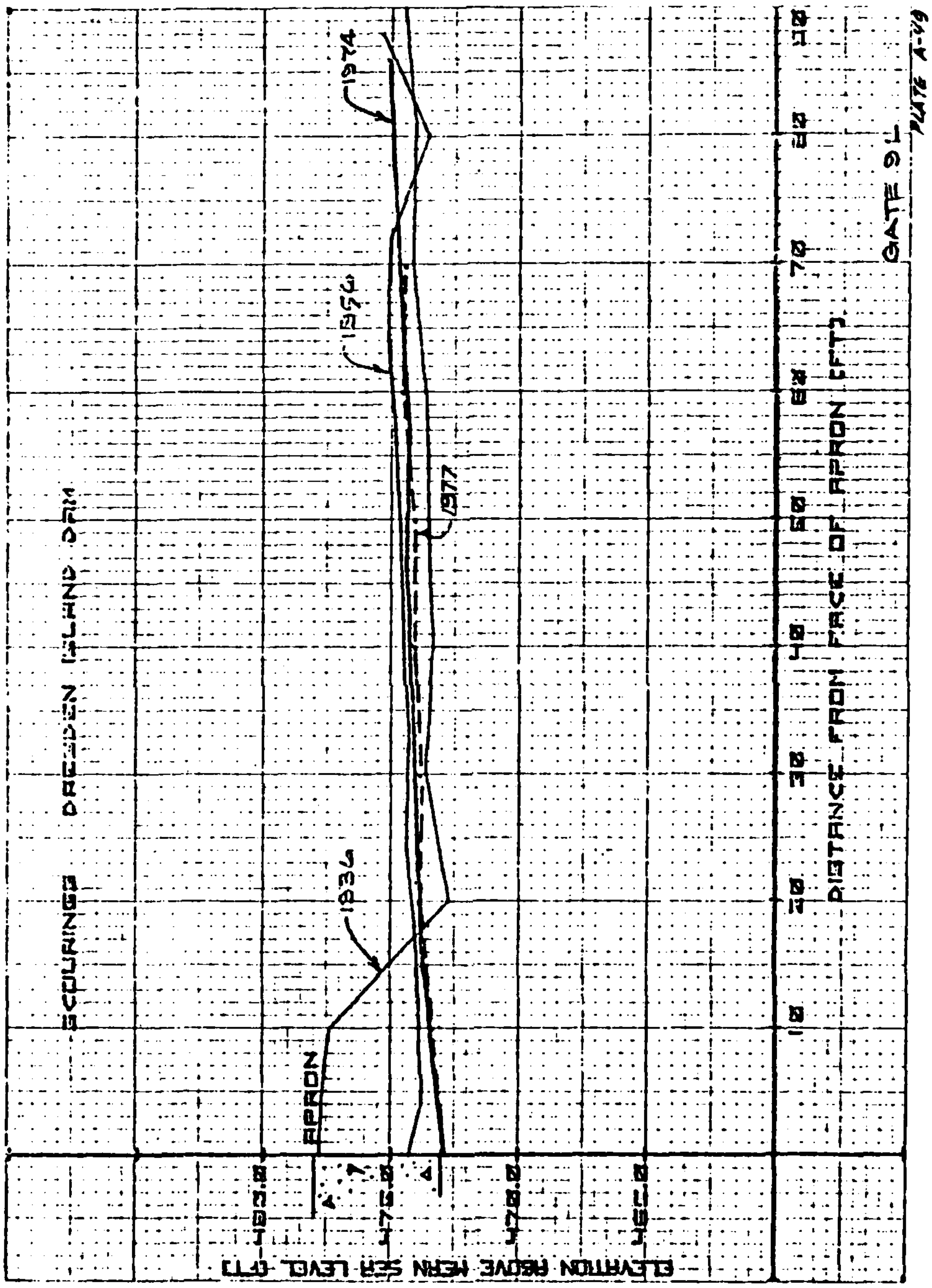
AD-A098 613 ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG--ETC F/G 13/13
CONCRETE AND ROCK TESTS, MAJOR REHABILITATION OF DRESDEN ISLAND--ETC(U)
MAR 81 R L STOWE, B A PAVLOV
UNCLASSIFIED WES/MP/SL-81-1 NL

3 of 3
AD
A098613

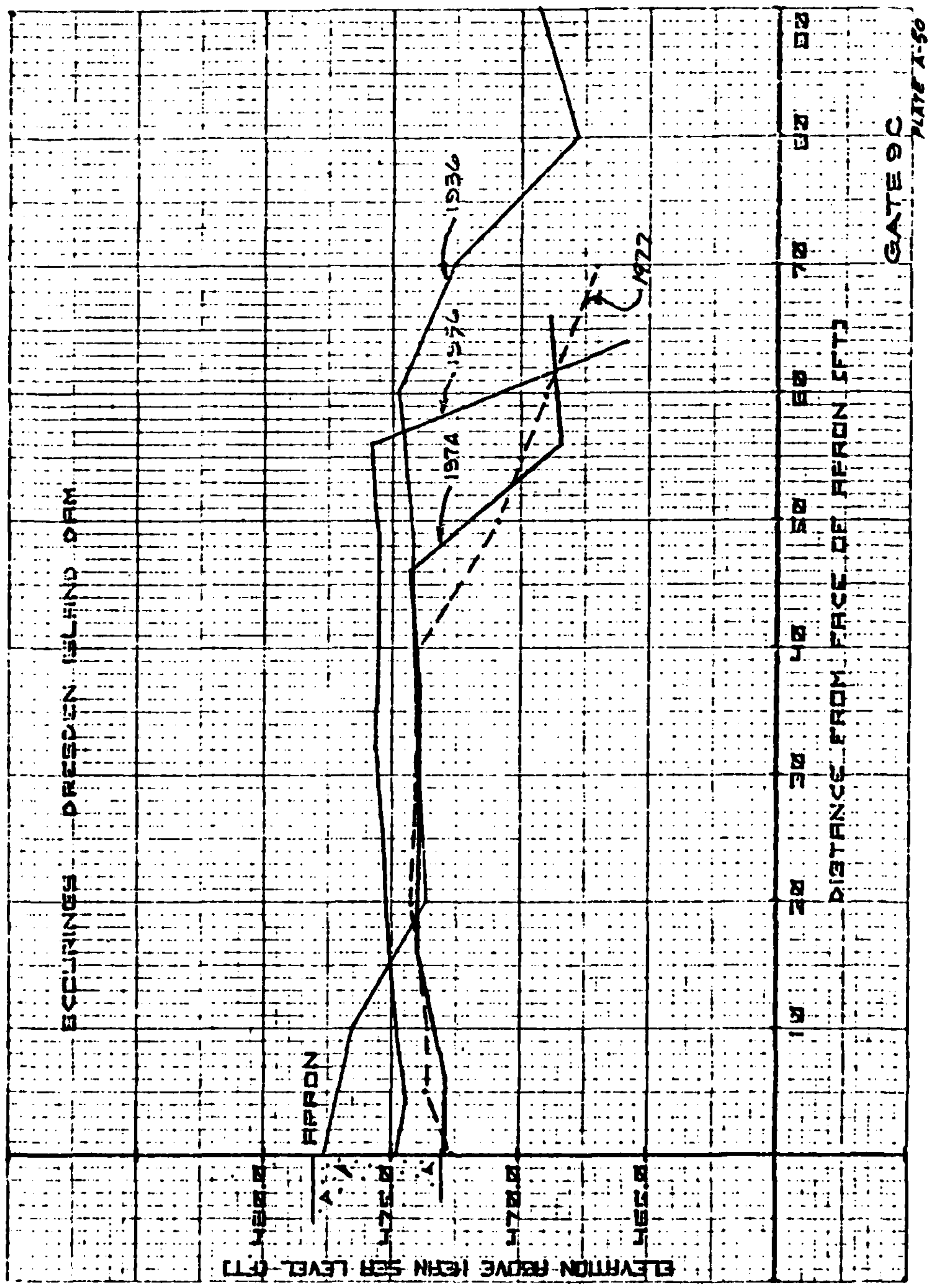
END
DATE
FILMED
6 81
DTIC

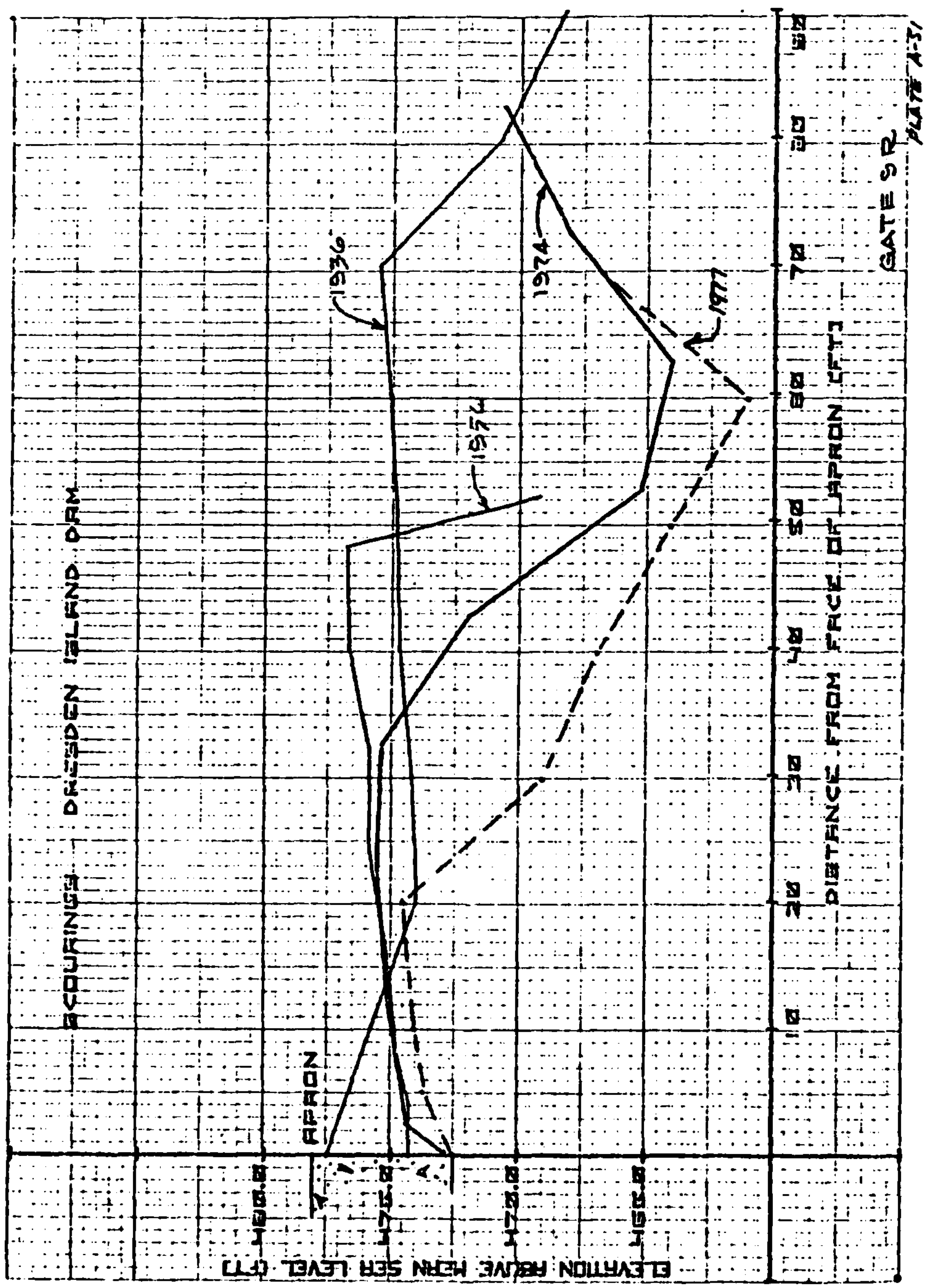


MICROCOPY RESOLUTION TEST CHART



SCOURINGS DRESDEN ISLAND DAM
ELEVATION ABOVE MEAN SEA LEVEL (FT)
480.0
475.0
470.0
465.0
APRON
1936
1956
1974
1977
10
20
30
40
50
60
70
80
90
DISTANCE FROM FACE OF APRON (FT)
GATE 9L
PLATE A-49


DRESDEN ISLAND DAM
GATE 9C
PLATE I-50
DISTANCE FROM FACE OF APRON (FT)
ELEVATION ABOVE MEAN SEA LEVEL (FT)
APRON
1936
1974
1972
480.0
475.0
470.0
465.0
10
20
30
40
50
60
70
80
90


SCOURINGS DRESDEN ISLAND DAM
GATE 9 R
PLATE A-31
DISTANCE FROM FACE OF APRON (FT)
ELEVATION ABOVE MEAN SEA LEVEL (FT)
APRON
1936
1954
1974
1977
10
20
30
40
50
60
70
80
90
100

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Stowe, R. L. (Richard L.)
  Concrete and rock tests, major rehabilitation of Dresden Island Lock and Dam, Illinois Waterway, Chicago District, Phase II compliance, scour detection : final report / by R. L. Stowe, B. A. Pavlov (Structures Laboratory, U.S. Army Engineer Waterways Experiment Station) ; prepared for U.S. Army Engineer District, Chicago. -- Vicksburg, Miss. : U.S. Army Engineer Waterways Experiment Station ; Springfield, Va. : available from NTIS, 1981.
  59, [35] p., [48] leaves of plates : ill. ; 27 cm. -- (Miscellaneous paper / U.S. Army Engineer Waterways Experiment Station ; SL-81-1)
  Cover title.
  "March 1981."
  Bibliography: p. 59.

  1. Boring. 2. Testing laboratories. 3. Dresden Island Lock and Dam, U.S. 4. Rock-drills. 5. Erosion. I. Pavlov, B. A. (Barbara A.) II. United States. Army. Corps of Engineers. Chicago District. III. United States. Army

Stowe, R. L. (Richard L.)
  Concrete and rock tests, major rehabilitation of : ... 1981.

Engineer Waterways Experiment Station. Structures Laboratory. IV. Title V. Series: Miscellaneous paper (United States. Army Engineer Waterways Experiment Station) ; SL-81-1.
TA7.W34 no.SL-81-1

DATE
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